COOLGARDIE WATER-SUPPLY.

BY

CHARLES STUART RUSSELL PALMER,
M. Inst. C.E.

WITH AN ABSTRACT OF THE DISCUSSION UPON THE PAPER.

EDITED BY

J. H. T. TUDSBERY, D.Sc., M. Inst. C.E.,
SECRETARY.

By permission of the Council.
Excerpt Minutes of Proceedings of The Institution of Civil Engineers.
Vol. clxii. Session 1904–1905. Part iv.

LONDON:
Published by the Institution,
GREAT GEORGE STREET, WESTMINSTER, S.W.
[Telegrams, "Institution, London." Telephone, "Westminster 51."]
1905.

ADVERTISEMENT.

The Institution as a body is not responsible either for the statements made, or for the opinions expressed, in the following pages.

LONDON: PRINTED BY WILLIAM CLOWES AND SONS, LIMITED,
DUKE STREET, STAMFORD STREET, S.E., AND GREAT WINDMILL STREET, W.

SECT. I.—MINUTES OF PROCEEDINGS.

28 March, 1905.

Sir GUILFORD L. MOLESWORTH, K.C.I.E., President, in the Chair.

(*Paper No. 3516.*)

"Coolgardie Water-Supply."

By CHARLES STUART RUSSELL PALMER, M. Inst. C.E.

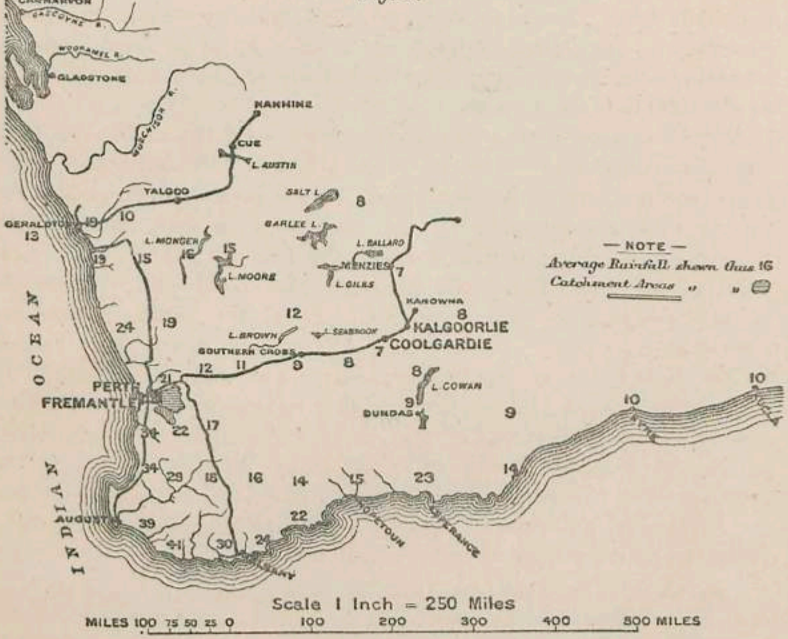
BEFORE proceeding to describe in this Paper the design and construction of the works undertaken for the water-supply of the Coolgardie district of Western Australia, it is necessary to touch briefly upon the history and topography of the district.

Since the discovery of the great inland goldfield of Coolgardie in 1892, the career of the State of Western Australia, which previously had made but slow progress, has been uniformly successful; for the resulting mining population created a profitable market for the agricultural and pastoral produce of the well-watered coastal country, which was therefore rapidly settled on as railway facilities were afforded. The town of Coolgardie is situated about 350 miles from the west coast and about 250 miles from the south coast (*Fig. 1*); and, although along the sea-shore and for a considerable distance inland this part of Australia is well watered, the portion—say, 300 miles by 250 miles—of the elevated tableland in the interior of which Coolgardie may be regarded as the centre is among the driest of the countries of the globe, the rainfall having been as little as 3½ inches in a year. Moreover, the surface soil generally is very porous and so excessively saline that, except in rock-holes after rain, really fresh natural water is practically unknown, although repeated boring has proved the existence here and there underground of small quantities of fairly potable water.

Coolgardie was discovered by pioneers who had pushed out, through this inhospitable country, for more than 200 miles from the terminus of the railway: they spread themselves over the length and breadth of the tableland already mentioned, discovering additional, though mostly smaller, goldfields. Their settlements,

however, were widely scattered, and the Government was soon faced with the serious problem of providing water for man and beast, not only at the mining-centres but also along the various tracks thereto. About £400,000 has been spent on smaller waterworks of every description, the exceeding dryness of the climate being soon made manifest by the poor result of each small but costly work carried out. It was thus proved that for any large supply of fresh water a source should be searched for elsewhere than on the surface or in the subsoil of any portion of this tableland.

Fig. 1.



The seriousness of the problem can be gauged by the usual prices paid for water in later days. Even after completion of these smaller works, the prices were:—25*s.* per 1,000 gallons when the occasional rains filled the tanks which formed one class of the works, and £4 per 1,000 gallons when only water condensed from the extremely salt fluid obtained in wells and shafts was available.

Yielding to the pressure of popular opinion, the Government spent several thousand pounds without result, in a bore-hole more than 3,000 feet deep in solid granite; various schemes for condensing on a very large scale, at the salt lakes situated in the goldfields,

were abandoned on proof of the excessive salinity of the water of the lakes, the difficulty and cost of obtaining a sufficient supply of even this water, and the high price of fuel; and then two proposals for conservation, with sources comparatively near to the goldfields, were considered but abandoned, as the low rainfall rendered it more than questionable whether the yield would be sufficient. By a process of elimination, therefore, there was reached the accepted solution of the problem, namely, a source in the Darling Ranges bordering the well-watered west coast (Fig. 2, Plate 1). This scheme had the additional advantage that all intermediate townships, as well as the adjacent Government railway, could be supplied from the main conduit, the railway being especially benefited in its course through about 250 miles of arid country wherein railway water-supply was known to have cost as much as £60,000 in a single year.

Scope and Character of Adopted Scheme.—By the middle of 1895 the Government of Western Australia had decided that some large comprehensive scheme would be necessary; and, orders for report and recommendation having been issued, there were prepared, under the instructions of the late Mr. C. Y. O'Connor, C.M.G., M. Inst. C.E., the Author's late chief and predecessor in the position of Engineer-in-Chief of the State, thirty-one alternative proposals, from which, after study, three were chosen to be placed before the Government. The source of supply in each case was to be an impounding-reservoir in the Darling Ranges, whence the water was to be pumped in successive lifts to Mount Burgess, north of Coolgardie: thence it was to be reticulated to the various mining-centres, of which Coolgardie was one. In Mr. O'Connor's Report, the three schemes were stated,[1] for comparative purposes, to be as follows:—

"The result of these calculations went to show (as for steel pipes) that, for one million gallons daily, the cost would be from, say, £700,000 to £1,000,000 (depending upon the size of the pipe), and with cost of delivery varying from 5*s*. 6*d*. to 8*s*. 6*d*. per 1,000 gallons; while, for five million gallons daily, the cost varied from, say, £2,200,000 to £2,700,000 (depending similarly upon the size of the pipe), with cost of delivery varying from 3*s*. 5*d*. to 6*s*. 7*d*. per 1,000 gallons; and that, for ten million gallons daily, the cost varied from, say, £3,500,000 to £4,600,000 (depending similarly on the size of the pipe), with cost of delivery varying from 3*s*. to 5*s*. per 1,000 gallons."

The scheme adopted was for a daily supply of 5 million gallons, at a probable capital cost of £2,500,000, and a selling-price of 3*s*. 6*d*.

[1] "Report on Proposed Water-Supply (by Pumping) from Reservoirs in the Greenmount Ranges," p. 8. *Perth*, 1896.

per 1,000 gallons, after allowing for interest and depreciation. The consumption of the water provided by this scheme, which is still in its infancy, has not yet amounted to more than one-fourth of the quantity allowed for; and the Author reported to the Government, soon after becoming responsible, that until much greater development of mining occurs, the consumption is not likely to exceed one-half of that allowed for. It is therefore due to those originally responsible to point out that when the proposals were inaugurated, and, in fact, up to the time when the works were opened, no information and no authoritative opinion outside the Public Works Department could be obtained as to the probable consumption; while, on the other hand, there were the greatest expectations in the public mind of more extensive working of low-grade mines, when comparatively cheap water should be available. These expectations were not confined to the general public; for, some doubts having been expressed in Parliament as to the probability of so much as 5,000,000 gallons being used daily on the goldfields, a well-known firm offered to take that quantity daily for 20 years at 3*s.* 6*d.* per 1,000 gallons, provided that the Government would not compete with them in price. In September, 1896, Parliament sanctioned the raising of a loan of £2,500,000 for the construction of a storage-reservoir of about 5,000 million gallons capacity, a 30-inch line of steel main throughout, and a series of eight pumping-stations, with the necessary receiving-tanks and distributing-reservoirs.

In January, 1897, a Commission of English engineers—consisting of Mr. John Carruthers (the Consulting Engineer for the State in London), Dr. George F. Deacon and Professor W. C. Unwin—was appointed to inquire into and make recommendations as to the kind, thickness and size of pipe to be employed; whether it should be placed above or below ground; and the number, positions and power of the pumping-stations and engines, and the pumping and break-pressure reservoirs. Mr. O'Connor, who was then Engineer-in-Chief to the State, personally placed all available information before the Commission, which issued two reports. In the first or interim report nine pumping-stations were recommended, as indicated in Figs. 3, Plate 1. In the final report the Commission submitted an alternative arrangement, with eight pumping-stations in lieu of nine; and in the adopted scheme the locations of the pumping-stations differ but slightly from those of the first eight stations proposed by the Commission in their interim report: but it was possible to omit the ninth pumping-station, as it was decided to deliver the water into a large service-reservoir at

R. L. 1630, near Bulla Bulling (Figs. 4, Plate 1), instead of on a high hill near Coolgardie, and to increase the lift at each of the last four stations by such small amount as would enable this to be accomplished.

A further advantage obtained from the appointment of the Commission was that full knowledge of the proposed scheme was obtained by the Consulting Engineer, thus enabling him to give his advice when sought from time to time, to make recommendations as to pumping and other machinery, and to undertake inspection of the material and plant exported to the State.

The detailed description of the works will be divided under the following heads:—

I. The storage-reservoir and its catchment-area.
II. The construction of the weir.
III. The pipe-line.
IV. The pumping-machinery.
V. The pumping- and service-reservoirs, reticulation, etc.
VI. Cost of the works.

The following general outline is given here to facilitate a clearer understanding of the details.

General Outlines of Scheme.—A daily supply of 5,600,000 gallons was provided for, of which 5,000,000 gallons was for use in the goldfields, and the balance for waste and consumption *en route*. The supply is obtained from an artificial reservoir, having a capacity of 4,600 million gallons. From this reservoir the water is pumped through a steel conduit, 30 inches in diameter, by a series of eight pumping installations, to the main distributing-reservoir at Bulla Bulling, 308 miles from the main storage-reservoir and 1,290 feet above the lowest outlet-level of the latter. From the Bulla Bulling distributing-reservoir the water gravitates for 21 miles to the Coolgardie service-reservoir, and thence to the Kalgoorlie service-reservoir, a further 23½ miles, the total length of the conduit from the supply reservoir being 351½ miles (Figs. 4, Plate 1).

The first pumping-station is located on the right bank of the Helena River and 650 feet down-stream of the storage-reservoir. The pumps draw their water from a stand-pipe 4 feet in diameter, which is placed immediately in front of them and is fed by a 30-inch steel main, which, beginning at the outer valve-house, passes under the boiler-house before entering the stand-pipe. The pumps here lift the water a net height of 415 feet, through 1½ mile of pipe, and deliver it into a concrete receiving-tank having a capacity of 468,000 gallons and a depth of 15 feet of water.

The pumps at Station No. 2 draw their water from this receiving-tank, the maximum suction-lift being $11\frac{1}{2}$ feet, and deliver it into a concrete regulating-tank at Baker's Hill, $22\frac{1}{4}$ miles from Station No. 2, the net lift being 340 feet. From the Baker's Hill regulating-tank, which is 15 feet deep and has a capacity of 500,000 gallons, the water gravitates to the West Northam regulating-tank, 12 miles distant. This tank is similar in construction to that at Baker's Hill, having the same capacity and depth. The net fall is 94 feet from Baker's Hill to West Northam, whence the water gravitates to the Cunderdin reservoir, a further 41 miles, thus making a total length of $75\frac{1}{4}$ miles between Stations 2 and 3. The Cunderdin reservoir has an available capacity of 10 million gallons. No. 3 pumping-station is located about $\frac{3}{4}$ mile from this reservoir, and the pumps draw their water from a stand-pipe, similarly to those at No. 1. The section between Stations Nos. 3 and 4 is $62\frac{3}{4}$ miles in length, the net lift at No. 3 being 215 feet. The water is delivered into a circular concrete tank at No. 4, having a capacity of 1 million gallons and a depth of 15 feet. From Station No. 4 the water is lifted a net height of 333 feet, and delivered through a section $32\frac{1}{2}$ miles long into a rectangular concrete receiving-tank 20 feet deep, with a capacity of 1 million gallons. At Stations Nos. 5, 6, 7 and 8, the arrangements are similar to those at Station No. 4, and the receiving-tanks at Nos. 6, 7 and 8 are similar in design to that of No. 5, having also the same capacity and depth. The net lifts at Stations Nos. 5, 6, 7 and 8 are respectively 52 feet, 106 feet, 56 feet and 183 feet, and the corresponding lengths of section 46 miles, $31\frac{3}{4}$ miles, 45 miles and $12\frac{1}{4}$ miles. From Station No. 8 the water is delivered into a main service-reservoir at Bulla Bulling, of 12 million gallons capacity. Thence the water gravitates to Coolgardie, and from Coolgardie to Kalgoorlie. These towns are provided with circular concrete service-reservoirs, that at Coolgardie having a capacity of 1 million gallons and that at Kalgoorlie of 2 million gallons.

Early in 1898 the first work on the scheme, namely, construction of the branch line of railway to the weir, was put in hand, and was completed in the following August. In April of the same year, excavation for the foundations of the weir was started; concreting was begun in February, 1900, the first pumping took place in April, 1902, and the weir and subsidiary works were practically finished in April, 1903. Contracts for the pipes were let in October, 1898, and for the pumps in March, 1900. The excavation of the pipe-trench was begun in March, 1900. The

laying and jointing of the pipes was begun in March, 1901; about 90 miles were completed that year and the remaining 260 miles (including the extension to Kalgoorlie) in 1902. The water reached Kalgoorlie in the middle of January, 1903, and the works were formally opened on the 26th of that month. The whole period of construction had thus been less than 5 years, although it was necessary to import all material for construction of the pipes, cement, valves and specials, lead for jointing, pumping-machinery, the ironwork in the weir, and much other material.

I.—The Storage-Reservoir.

When the scheme was first propounded, and, in fact, until shortly before the construction of the weir was begun, there were no river-gaugings available: consequently, in judging of the probable inflow into a reservoir, it was necessary to base calculations on results obtained in other countries. About 3,000 square miles of the Darling Ranges having been examined, and thirteen possible sites surveyed in a preliminary manner, it was finally decided to place the reservoir at Mundaring on the Helena River, where the cost of construction per million gallons of storage would be least. Fig. 2, Plate 1, shows the catchment-area and the rainfall-records available; and notwithstanding that the catchment-area is 569 square miles in extent, it was decided to provide storage sufficient to meet 2 years' demand and loss.

On the face of it this was an excessive allowance, especially when it is considered that to fill this large reservoir there is required an off-flow of what would usually be considered a very small fraction—only 3 per cent.—of a rainfall of 18½ inches, which is less than the average of the minimum yearly precipitation at Mundaring and York. But the country in which the upper reaches of the Helena River are situated is formed of crystalline rocks, generally covered over large areas by ferruginous conglomerate, and, in a measure, by loamy sand, which in places extends to a depth of 20 to 30 feet below the surface. The conglomerate and sands generally overlie kaolinized granite, which, in turn, merges into solid granite. In the vicinity of the weir, the rock is more exposed, the country is less flat, and the ranges are better defined. The whole of the water-shed is thickly timbered with jarrah, red gum and wandoo, the jarrah predominating on the lower, and the wandoo on the upper reaches. Besides this heavy timber, the country is closely covered with an undergrowth of "blackboys" and other scrubby plants.

The actual yield from the catchment-area, therefore, is shown in the following Table, which gives the discharge of the Helena River since gauging was undertaken.

DISCHARGE OF HELENA RIVER AT WEIR SITE.

	Mean Rainfall Mundaring and York.	Discharge.	Ratio of Discharge to Rainfall.
Year	Inches.	Million Gallons.	Per Cent.
1897	24·5	672	0·34
1898	30·76	3,802	1·50
1899	27·17	1,857	0·83
1900	33·25	9,622	3·50
1901	25·0	1,401	0·69
1902	19·3	323	0·20

Not only are these figures very low, but the ratio of the discharge to the rainfall varies considerably more than does the rainfall. The small results as a whole can be accounted for partly by the absorptive nature of the soil of much of the catchment-area, therein differing from the catchment-areas usually available in other countries, and partly by the fact that the rain is precipitated very unfavourably: for although the annual fall, in the vicinity of the reservoir for instance, averages about 37 inches, it is spread over a period extending from about May to November, inclusive. During some months, it rains nearly every day; but only on very rare occasions does the fall exceed 1 inch in 24 hours, the average being less than $\frac{1}{4}$ inch, generally in light intermittent showers. The result is that the main watercourses do not begin to flow until 10 to 12 inches of rain have fallen, and they stop almost immediately the rainy season ends. The rainfall for the year 1902 may be taken as typical of the rainfall generally. During that year the total rainfall, as recorded at the Helena weir, amounted to $27\frac{1}{2}$ inches, the total number of rainy days being 81; *i.e.*, the average precipitation per rainy day was only 0·34 inch. The maximum rainfall in any one day was 1·41 inch.

The unusual variations in the yield are due, in the Author's opinion, to two other causes, whose effects in new countries where records are scanty require much experience and consideration for their correct estimation. The first is that the rainfalls of York and Mundaring, which are all that are available, require to be greatly discounted, owing to a rapid rise of the country for several miles inland from Mundaring, and then a fall to the tableland of the interior; and it is therefore probable that the rainfall on the

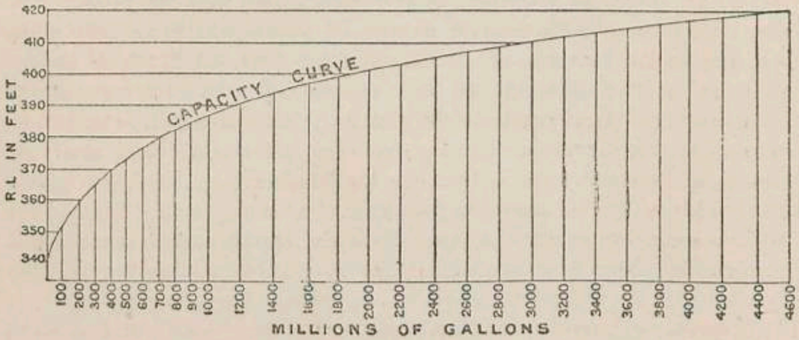
reverse slope of the Darling Ranges, and on much of the tableland adjoining, is less even than that at York. The second cause is that on the tableland, where the rainfall is smaller and the country is more absorptive than on the ranges, a large and more or less definite amount should be deducted from the rainfall before any estimate of the off-flow can be made. This opinion is supported by Tables I and II in the Appendix (p. 50). Table I contains the results of gaugings at two points lower down the same river as that on which the reservoir is situated, with, for the sake of contrast, the results obtained from the catchment of a selected portion of the inland country which is adjacent to that which drains into the Helena. It also gives the results of gauging the Canning River at a spot where, although the catchment includes a larger proportion of quick-shedding ground than in the case of the Helena at Mundaring, the result, by no means apparent at first, is that the average yield of the total catchment is better than that of the Helena, although only half the size. Moreover, after the exceedingly low yield of 1902, the Author had weirs constructed across the several streams entering the reservoir-basin, and the results of gaugings are given in Table II. These results show conclusively the great conservative care necessary in dealing with questions of this kind in countries so dry as are parts of Australia, and they also justify the abnormal storage provided in the present case.

It was early evident that much of this difficulty would be obviated by placing the weir lower down the river (Figs. 2, Plate 1), and thus including a larger proportion of quick-shedding catchment. This, however, would have given a worse reservoir-site (*i.e.*, less impounding-capacity for the same expenditure), and would have entailed extra expense, both capital and annual, on account of the greater pumping-lift: the site higher up-stream was therefore adhered to. The startling results of 1902, however, could not be foreseen; and as, in the Author's opinion, the available records of rainfall (Table III) render it more than doubtful whether, when the full estimated daily consumption is reached, even the 2 years' storage-capacity will be sufficient to tide over bad seasons, the responsibility was incurred by him of recommending any necessary additions to the works. It was not considered advisable to raise the reservoir-wall —although it is strong enough to bear some additional static pressure —since the diagram of capacities (*Fig. 5*) shows that the limit of economy of the site has already been reached; and it has therefore been recommended that so soon as the demand warrants that course, catch-water drains should be extended into the well-watered and

quick-shedding country draining into the Helena below the present weir. By this means, for an expenditure of about £45,000 (only 1½ per cent. of the total cost of the scheme), enough extra catchment-area could be included to ensure a yield from the present reservoir of 6½ million gallons daily—say 50 per cent. more than what is now a certainty.

Evaporation and Loss from Reservoir.—As will be explained later, the site of the weir was so badly fissured, and the basin of the reservoir so extensively crossed by basaltic dykes, that competent geological authorities believed that extensive stopping with concrete might be necessary. It was a matter of great interest, therefore, to ascertain what loss, if any, from the reservoir might result from this fractured condition; for it was decided that, except as regards the fissure across the site of the dam, no precau-

Fig. 5.



tionary measures should be taken in the first instance. Between the 1st November, 1901, and the 28th February, 1902, no water at all was drawn, and a favourable opportunity for testing the probable annual loss was thus afforded. The results are shown in Table IV of the Appendix, and, as the total loss is at the rate of only 4½ feet of depth per annum—which is not much more than the evaporation alone should amount to—the danger of building where similar basalt dykes occur can be seen to be not excessive. The point is one of the greatest interest in Western Australia, not only because the location generally of the streams in the ranges seems to have been determined by the greater ease with which these basaltic dykes have been disintegrated and eroded, but also because practically the only site available for the reservoir of the ultimate gravitation scheme for the West Australian metropolitan area, lately

reported on by the Author, is even more fissured than that at Mundaring.

Quality of the Water.—Except in a few tracts where salts have been leached out of the soil by heavy rainfall, even the surface waters of Western Australia generally, although soft, contain a comparatively large amount of dissolved mineral matter, chiefly common salt. High chlorine results have therefore to be expected on analysis, and this test of possible contamination of the sources of supply, whatever its value elsewhere, is inapplicable; but as the catchment is so large, and as much of the area was alienated before the scheme was inaugurated, and would cost an excessive sum to buy back, a small amount of habitation on the catchment is inevitable. Moreover, except in these small patches, the land is covered with forest and scrub; and, with a small ratio of off-flow, the decaying vegetable matter would be expected to result in high ammonia figures on analysis of the water. Although this would in itself be harmless, it was recognized that the presence of the dissolved organic matter might be accompanied, on occasion, by dangerous bacteria of decomposition; and, as filtering before delivery is not included in the scheme, it was determined to institute periodical analyses in the first instance, and, later on, to make also regular bacterial examinations. The results of analyses made in February, May, October, and December, 1903, are given in Tables V–VIII, and disclose the fact that, owing apparently to anaerobic action in the pipes between the storage-reservoir and Coolgardie there is a marked improvement in the quality of the water.

Disposal of Surplus Flood-Waters.—Although when the scheme was inaugurated there were no gaugings by which the probable total inflow into the reservoir from year to year could be arrived at with certainty, there was ample evidence to show that the Helena and other streams similarly situated were liable to heavy floods; and as at the site chosen for the weir the valley through which the Helena runs converges abruptly, being in fact a deep gorge flanked on both sides by high hills, the width at the bed of the river being only 15 feet, and at 100 feet above the bed 750 feet only from solid rock to solid rock, the usual method of disposing of flood-waters, namely, by means of a by-wash, was precluded by its cost. It was therefore decided to pass all floods over the weir-crest, notwithstanding that calculation showed that for safety as much as 5 feet in depth over the whole length of the weir would have to be allowed for. Although, so far as the Author is aware, the weir is the highest overflow-weir in existence, this depth was

not considered excessive, because no debris whatever is brought down in floods, which, even when of the heaviest, could not be of very long duration. Indeed, the whole flow of the river lasts less than half the year at most, so that sufficient time is afforded for repairing any damage done to the weir-face and footings. In order to facilitate the descent of the water, the profile of the weir-crest was approximated to a parabola, and the form of the curve follows very closely that of the Holyoke Dam, in the United States, which was determined by experiment. The reservoir overflowed for the first time during the rainy season of 1903, and, for about 2 weeks, 6 inches of water flowed over the crest without doing any damage to the wall.[1] The water clung perfectly to the whole wall-face while descending. The same result is not to be expected when the depth over the crest is much greater; but obviously some departure of the deeper flowing water from the wall would not matter much. The down-stream face of the dam is broken by three guide-walls which prevent any scour at its foot by the spill-water, which would otherwise have run along the toe; and where the wall is highest, a spill-water basin or water-cushion has been provided, from which a wide channel, excavated in the river-bed and lined with stone, carries all flood-water rapidly away clear of the dam and of the pumping-station below it.

The loss by evaporation and percolation has proved to be very small, and the overflow and draw-off arrangements of the reservoir have worked most satisfactorily.

II.—Construction of the Weir.

The weir was built to the section shown in Figs. 6, Plate 1, the governing factors of the design being that the maximum pressure on any portion of the wall should not exceed 8 tons per square foot, and that the centres of pressure should be well within the middle third, both with the reservoir empty and when 5 feet of water was flowing over the crest.

Preliminary Works.—The reservoir and the first pumping-station are situated at the bottom of a deep valley some miles from the nearest railway, and as all material, except stone for the concrete of which the weir was built, had to be brought from a distance, the first work put in hand was the construction of a tram-line, to the standard railway-gauge of the State, starting from Mundaring

[1] The Author has heard since the above was written that 1 foot 6 inches in depth of water passed over the crest in the beginning of August, 1904.

station on the existing line of railway. The next question was the provision, at a comparatively waterless spot, of a permanent supply of water fit for the use of the many men to be employed, as well as for the works. The requirements were met by constructing, in the bed of the future reservoir and about 9 chains above the weir-wall, a temporary concrete dam impounding about 20 million gallons, and by forming, from the by-wash with which this small reservoir was provided, a channel capable of carrying away 100 million gallons per diem. The channel was formed for the most part of open cut, but a timber flume carried the flood-waters across the weir-site.

Foundations.—Generally speaking, the country at the reservoir site is very rocky, consisting largely of undecomposed granite, traversed by intrusive basaltic dykes whose direction is mostly at right-angles to the course of the river. At the site of the weir, however, the granite showed out particularly clearly, and the few trial-shafts, put down where the rock was shattered, reached solid granite at no great depth, the deepest of the shafts being only 20 feet deep from the ground-surface. On opening up the foundations, however, it was discovered that the rock was nothing like as solid as surface indications and trial-pits promised; for on the right bank a large portion of what at first appeared to be bed-rock was found to consist of an immense boulder with a large cavity below it; and under the bed of the river the granite was very badly fissured over the full width of the foundations. It was not possible to vary the site, as the disruption was found to extend both up- and down-stream for a considerable distance and there was no alternative but to follow the fissure down, which was done until a depth of 90 feet below river-bed was reached. At this level the filling material in the fissure was found to be so compact that it was but slightly eroded by a jet of water discharged under a pressure of 250 lbs. per square inch. It having been seen that the fissure had a northern underlay, a vertical boring was now made on the north bank of the river, which cut the fissure at about 165 feet below river-bed, and was continued for a further depth of 52 feet. The bore was then filled with water, and the material in the fault was subjected to a hydrostatic pressure equivalent to a head of 690 feet, which was maintained for 4 hours, during which time the foot- and hanging walls of the fissure, and the line of fissure at the bottom of the excavations, were all carefully examined; but no signs of moisture could be detected. It was concluded that the material in the fissure, at the depth which the excavations had then attained, was

impervious to water, and that it would therefore be safe to erect the weir thereon.

Where the wall would be highest, that is, where the fissure occurred in the foundations, the excavations were carried down about 15 feet from the building-line in a vertical direction on the up-stream face; but as one of the basaltic dykes crosses the valley a short distance away on the down-stream side, it was considered necessary to remove the whole of the material between the hanging wall of the dyke and what would otherwise have been the toe of the weir. The concrete filling of foundations was carried up to bed-level on the up-stream face, but on the lower side the mass filling was stopped 18 feet below bed-level and the wall proper was begun to the designed section. The granite beds, or floors, were deeply chased in longitudinal rows about 6 feet wide and 3 feet deep, and the toe of the wall-batter, where it met the granite floor, was channelled the whole length to key the concrete in.

The great inequality in the depth of the foundations, and their apparent doubtfulness for a work of this magnitude, have not appreciably affected the weir; for although very fine vertical lines, such as invariably occur in the concrete lining of service-reservoirs in hot countries, have made their appearance here, they have not extended, and any slight sweats have taken up.

Drawing-off and Scouring Arrangements.—The reservoir is provided with two valve-towers constructed of concrete. The inner tower (Fig. 7, Plate 1), situated on the reservoir side of the weir, was built into, and concurrently with, the main wall, being approximately semi-circular in section. The outer valve-tower (Figs. 8 and 9, Plate 2) is rectangular in section, and is situated 175 feet down-stream from the centre of the weir-wall, being connected therewith by a viaduct, which carries the outlet- and scour-pipes, all solidly bedded in concrete, as far out as the outer valve-house. Ingress to the inner valve-tower is obtained by means of a steel gangway running over the crest of the weir, and supported thereon by granite cut-water piers 52 feet apart between centres.

Provision is made for drawing water from the reservoir at three different levels, namely, at 25½ feet, 53 feet and 80 feet below full-supply level, by means of 24-inch cast-iron bell-mouthed pipes, passing through the valve-tower wall into a cast-iron stand-post. Each draw-off inlet is provided with a stop-valve placed in the valve-chamber, from which valve-rods are carried up to bevel-geared headstocks, all placed on the upper valve-tower floor, which

is 1 foot 9 inches above maximum flood-water level of the reservoir.

Over each inlet are placed screens which can be removed for cleaning by means of chains worked by winches carried on an outer platform running round the valve-tower, and supported therefrom by brackets. Two 24-inch cast-iron spigot-and-faucet outlet-pipes pass from the stand-post, at 80 feet below full-supply level, through the weir-wall to the outer valve-tower. Each outlet is provided with a stop-valve in the inner valve-tower, and these are regulated similarly to the valves on the reservoir-inlets.

A 30-inch scour-pipe, leading from a fore-bay 90 feet below full-supply level, runs through the inner valve-tower, and through the weir-wall into the outer valve-tower. It is provided with a stop-valve in the inner tower, which is worked by a worm-geared head-stock placed on the upper floor. From the outer valve-tower the scour passes into the river, where it has its discharge. Both the outlet-pipes and the scour-pipe are provided with valves in the outer valve-tower, which will be brought into use only in the event of accidents to the regulating-valves in the inner valve-tower. Any water soaking through the wall of the inner valve-tower is led into a sump, whence it can be lifted into the scour-pipe by means of a water-ejector, supplied with pressure water from the lowest inlet. At the outer valve-tower, the two 24-inch outlet-pipes junction into a 30-inch pipe, which runs to the stand-pipe in front of the engines at No. 1 station.

The details of all the ironwork used in the construction of the weir were drawn out in the State, and all ironwork was obtained from Great Britain. It speaks well both for the accuracy displayed in the preparation of the drawings, and for the care exercised in the manufacture of the various appliances, that when being grouped together as the work progressed, all parts fitted correctly into their respective places, without any alteration whatever.

The site of the reservoir, about 800 acres in extent, was grubbed and cleared, and all fallen timber and decaying vegetable matter was taken out of the river-bed and burned; later on the suckers and scrub were again cut down and burned. About 20,000 acres of the lower catchment-area was ring-barked with the object of increasing the inflow.

A concrete-lined spill-water basin, about 150 feet long by 100 feet wide, is constructed in the bed of the river, at the toe of the

wall, with a depth of water of about 10 feet. The water is confined by a mound across the river-bed constructed of rubble faced with concrete.

The excavations for the foundations were begun in May, 1898, and on their completion in January, 1900, the building of the wall was started, and was carried on both day and night until completion in June, 1902, an electric-lighting plant and eight arc-lamps placed at points of vantage affording ample light for operations by night.

Cement and Concrete.—In the construction of the weir-wall 76,418 casks of cement were used, and a further 1,090 in the spill-water basin and other subsidiary works, or a total of 77,508 casks, of which 19,767 casks were of German manufacture and the balance British. The German cement was chiefly used in filling the deep excavations made in sinking on the fault in the bed of the river.

The length of the average passage by steamer from London to Fremantle was more than 6 weeks, and by sailing vessel 90 to 100 days; and as on arrival in the State the cement was received into storage-sheds where it lay at least 1 month, but generally for a longer period, during which time tests were made preliminary to its despatch for use, the cement had some chance of losing any "freshness" which it might have had when first placed in casks, and needed comparatively little slaking. A cement which did not demand much slaking before use was especially necessary in connection with the smaller scattered works of the scheme, distributed as they were over 350 miles, and mostly in country whose dry atmosphere would not tend to satisfactory, or, at any rate, speedy, slaking. In these smaller works, the cement having passed the necessary tests, was used direct from the casks, because to slake and then repack it would have entailed incommensurate expenditure; but at the weir provision was made for slaking fully all cement requiring it.

The tests, which were of an exhaustive character,[1] were directed not only towards determining the qualities of each batch of cement, but also to so ascertaining those qualities that after slight treatment in the State parcels which seemed at first to be doubtful might be used without hesitation. Situated, as the works were, at such a distance from the source of supply, this was essen-

[1] In the year 1902 alone over 9,000 briquettes were made, not only for immediate use, but also to be broken for comparison, 3 months and 6 months and 1, 2 and 3 years after making.

tial. Taken altogether, the cements received were very satisfactory, and as the long-date tests become due, and the samples are examined and the briquettes broken, the results confirm the good qualities adjudged after the shorter tests. In all, ninety-two complete analyses were made, of which those in Table IX. (p. 54) may be accepted as average results. The specific gravity varied between 3·05 and 3·13. On the whole, the cement used was exceptionally well ground, that received towards the completion of the works being even finer than the earlier consignments. After the works were begun, a special set of bulk tests was carried out. Several casks of the different brands were sampled, and 25 lbs. of each brand was carefully passed through sieves with a mesh of 14,400 holes per square inch. The residues were—

German cement	7·04 per cent.
English cement	12·79 to 13·40 „ „

The tests of tensile strength ranged from the usual 3-day and 7-day hot- and cold-bath to 28-day cold-bath tests, a reserve of briquettes being often retained from the various batches for long-date tests. As a rule the results were very good, even when, fresh from the cask, the cement was subjected to the hot-water treatment. The hot bath was utilized to determine the necessity for slaking, it being found that a cement which showed a falling-off from the cold-bath results, when treated for a similar period in hot water, generally headed the cold-bath records after being fairly slaked: and there are numerous series of tests showing satisfactory increase in tensile strength at various dates up to 12 months. Cements, however, which showed a tendency to fall off in strength in hot water had to give undeniably good results after the requisite periods of slaking, before being despatched to the works.

One feature in the relative rise in temperature on hydration of slaked and unslaked cement is worthy of notice. On several occasions samples taken straight from the casks showed a comparatively small rise in temperature, yet the same cement, after exposure to the air, registered a considerable rise. For the purposes of closer examination a long series of experiments was made with the same shipment of cement air-slaked under three different conditions, namely:—

(1) Under the corrugated-iron roof of a shed.

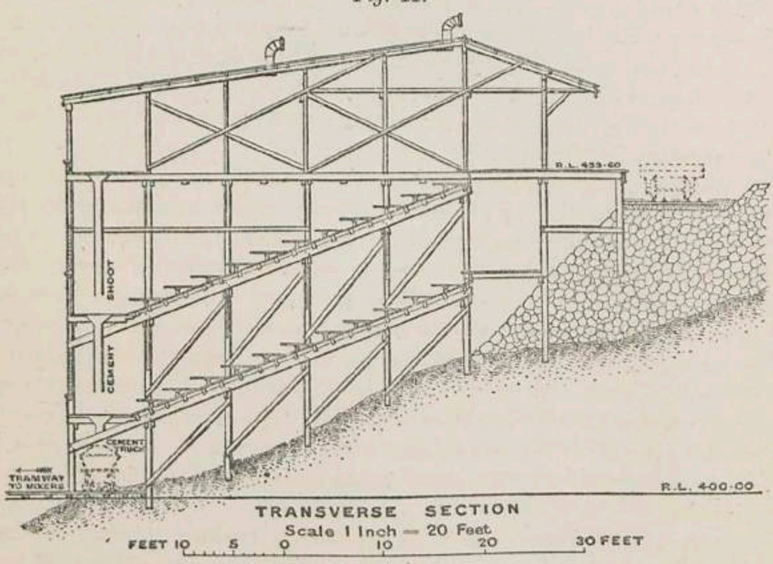
(2) Under the wooden floor of the same building, spread in thin layers on a large tarpaulin and turned over daily; and

(3) Under the same floor, but placed in a box covered with wet

bags, passages for currents of air being left between the cement and the bags, which were wetted and turned daily. The maximum rise in temperature is shown in Fig. 10, Plate 2. It is difficult to assign reasons for these results, but Table X. in the Appendix, obtained from long-date tests of this same consignment, shows satisfactory increase of strength.

The effect of slaking on subsequent expansion was also the subject of a long series of tests. Ordinary glass test-tubes, 6 inches by 1 inch, were filled with stiff grout, and placed in cold-water baths after the cement had set. The tubes usually cracked after 3 days or more when filled with fresh cement, showing a high rise of

Fig. 11.



temperature on hydration; but as slaking proceeded, so did the energy of the cement decrease. Further, there was apparently renewed activity after months of quiescence, tubes being cracked although the cement core itself remained hard and sound and, so far as the eye could detect, absolutely unharmed.

The receiving- and slaking-shed for the cement (*Fig. 11*) was built at the temporary end of the tram-line, that is, adjacent to its crossing of the weir-site, the upper floor of the shed being at the same level as that of the railway-wagons. The storage-capacity of the shed was 2,000 casks, and one-half of this quantity was taken by the slaking-tables, arranged on a falling gradient

so that one tipped on to the next and so on, until the cement arrived at a shoot leading on to the trucks, which conveyed it to the mixing-shed. A large portion of the cement, however, did not require special air-slaking. This, after having been put through a fine-meshed sieve, passed down a shoot to the concrete-mixer without being placed on the slaking-tables. Below the cement-shed, on the same level as the quarry-road, the stone-crushing plant was erected. It consisted of a No. 4 Gates crusher capable of dealing with 20 cubic yards of granite per hour, and driven by a 25-HP. Robey engine. In the same shed, but below the crusher, was situated the concrete-mixer, portable and self-contained, of the rotating-barrel type, mechanically fed with cement, stone and sand in the proper proportions, and capable of mixing 20 cubic yards of concrete per hour. All this machinery, from the cement-shed to the delivery end of the concrete-mixer barrel, was designed so that every operation which could be effected with advantage, or could be helped, by gravity, was so arranged; and the whole proved very satisfactory in working.

Except about 1,000 cubic yards of sharp, coarse-grained sand obtained from the river about 1 mile below the weir-site, the whole of the sand was brought from either Lion Mill or Bayswater, distant 8 miles and 22 miles respectively, by railway. That from the former was of quartz, and very fine-grained, yielding even and good results in the testing-room. The quarry, however, required heavy stripping of mould, and the sand itself required screening and thorough washing, to cleanse it from vegetable and earthy matters. This entailed the erection of a sand-washing plant. The sand from Bayswater was of a much better class, and required but light washing to free it from all earthy material.

About 30,000 cubic yards of spalls, for crushing to concrete size, were selected from the material obtained in the excavation of the foundations. For the plums and the balance of the spalls required, a quarry was opened on the north bank of the stream, below the weir, and about 70 feet above river-bed.

The weir and all accessories were built of concrete, but in the former, large rough granite blocks, just as quarried, were introduced into the concrete. It was originally intended to build the wall with 50 per cent. of these large blocks; but without proper plant, which was not available, handling would have been very expensive. The concrete consisted of 5 parts by measurement of granite crushed to $2\frac{1}{2}$-inch gauge, 2 parts of cleaned sand and 1 part of Portland cement. So long as the wall remained below the

level of the mixing-house, the mixture was conveyed to the work on an endless conveyor working in a trough, with travelling boards secured by ropes and spaced 2 feet apart, thus ensuring that the heavier aggregate was not separated from the matrix on the way. Later, the concrete was conveyed on a trolley-line in skips, to a large derrick-crane, which lifted the skips on to temporary tram-lines on the growing wall: they were then pushed by hand to a travelling steam-crane which lifted each skip in a bridle, overturned, emptied, and returned it to its carriage. The concrete was spread and rammed by hand, the various layers being broken up so far as the width of wall would allow, in order to break bond in both beds and joints; and in addition, the large rough blocks, up to 2 cubic yards in volume, were deposited and thoroughly bedded and grouted, in order to key the bedding-planes together.

For the first 10 feet the batter was lined and the concrete retained by rubble masonry, but above this level wooden framing was substituted. This framing was of Oregon pine, and consisted of uprights 9 inches by 3 inches, and 15 feet long, cut to the sweep of the wall section, spaced 3 feet apart and closely lined on the wall face with 12-inch by $\frac{1}{2}$-inch Oregon boards. For the first few feet upwards, the studs were held in position by shores, but later they were bored for 1-inch bolts about 18 inches long, at vertical intervals of 3 feet. Each bolt was fitted with an 8-inch by 3-inch by $\frac{3}{4}$-inch iron screwed washer-plate, which was built into the concrete, and remained there after the bolts were withdrawn and the holes grouted. Each vertical stud was lap-jointed and bolted to the succeeding one, the lap being sufficient to allow of two bolt-holds in the concrete before the lower boards were removed. No cross stays or ties were used across the wall, and the front and back framings were independent. The uprights were aligned throughout with a theodolite, the heads of each section being cut off to the required level and fixed to the width of wall corresponding with that level, with an allowance for outward pressure of the wet concrete. Rendering of the face was not desired or found necessary, as great care was taken, when depositing the wet concrete in contact with the moulding-boards, to keep all stone well back with straight spades, and a good finished face resulted on stripping. The valve-tower and viaduct, which were carried up with the main wall, were similarly built between moulding-boards, the frames inside and out being formed of upright studs, cross silled and lagged with 4-inch by $1\frac{1}{8}$-inch tongued and grooved Oregon boards fixed vertically.

III.—The Pipe-Line.

The points on which the Commission of English engineers were asked to advise were, as regards pipes and main generally—

(*a*) "Whether the pipes should be laid in a trench and covered in, or left exposed to view."

(*b*) "Whether it would be safe to rivet up the whole line of pipes, or whether joints, to allow for contraction and expansion, are necessary; the kind of joint most suitable, should they be necessary."

(*c*) "Material and method of manufacture of pipes, whether welded or riveted, and whether welding and riveting shall be square or spiral." The use of cast iron being prohibited by the cost and the difficulty of freight both by sea and by land, the Commissioners were not to take it into account.

(*d*) "The diameter and thickness of pipes, and method of protecting."

As regards (*a*), the Commissioners were informed that there were possibly deleterious salts in the soil of a large part of the district through which the aqueduct would pass; and, for this reason, and also in order to avoid pressure on the empty pipes, to save the expense of trenching, and especially to facilitate detection and suppression of leakage, they recommended that the pipes should be laid above ground, uncovered, with expansion-joints.

The Commission recommended that the pipes should be of steel throughout, supported on bolsters, and riveted up in lengths of about 110 feet, with expansion joints at these intervals, and anchor joints midway, fixed to masses of concrete or piles, in order to prevent the pipes from creeping. The minimum thickness was fixed at $\frac{3}{16}$ inch; and the pipes were to be longitudinally riveted where the pressure was such that the thickness of shell for riveted pipe was not required

Class of Pipe.	Number of Lengths of Pipe.					
	27½ feet long with plates ¼ inch thick and of internal diamete.			28 feet long with internal diameter 31 inches at larger end and reduced at smaller end according to thickness of plates to form telescopic joint.		
	26 Inches.	27½ Inches.	29 Inches.	Of $\frac{3}{16}$ Inch Plate.	Of ¼-Inch Plate.	Of $\frac{5}{16}$-Inch Plate.
Welded . .	5,592	4,393	5,710	..	..	..
Riveted . .	..	..	..	23,425	19,759	3,257

to be greater than $\frac{1}{4}$ inch, and welded for all higher pressures; with a minimum thickness of $\frac{1}{4}$ inch.

Tenders for the pipes were accordingly invited from Australia, Europe and America, the quantities specified for being as shown in the Table on p. 23. Tenderers were at the same time invited to submit alternative prices for any other kind of pipe which they desired to put forward. The lowest of the tenders received were as follows the prices being for delivery in the Colony at a point 22 miles inland:—

Class of Pipe.	Lowest Tenders received in	
	Europe.	Australia.
	£	£
Riveted pipes	782,708	682,827
Welded "	472,600	..
Locking-bar pipes in lieu of welded pipes .	..	239,868
Total	1,255,308	922,695

The locking-bar pipe, for which alternative tenders were received, had been considered by the Commission and favourably commented on, but was not recommended for so large a scheme, because proof of its successful manufacture and use on any considerable scale was not then available. Subsequently, however, and before receipt of the tenders, 10 miles of main, $25\frac{1}{2}$ inches in diameter, had been laid in South Australia. It had been found that pipes made from $\frac{1}{4}$-inch plate and fresh from the closing-machine would withstand a pressure of 400 lbs. per square inch—or nearly twice what would be allowed continuously in practice on pipes of this thickness of plate—without a weep; and, moreover, all pipes which did not stand the test could be passed back to the closing-machine to be reclosed, instead of being subjected to the usual caulking-processes so injurious generally to the plates and jointings. Practical use on a fair length of main also showed that the jointing could be successfully accomplished, thus leaving only questions of comparative cost and comparative usefulness to be considered in deciding whether the new pipe should or should not be used in place of welded and riveted pipes.

Taking first the Australian prices for locking-bar pipes and contrasting them with those for welded pipes, the saving is seen to be very marked, being within a few pounds of 50 per cent. Moreover, the price of locking-bar pipes was but little more than

that of riveted pipes. The lowest tenderers were therefore asked to consider the matter again, and they quoted prices for the locking-bar pipes which contrasted as follows with those received for the riveted pipes:—

Thickness of Metal in Pipe.	Riveted Pipe.	Locking-Bar Pipe.
Inch.	£ s. d.	£ s. d.
$\frac{3}{16}$	12 12 9	13 10 0
$\frac{1}{4}$	16 5 0	16 15 0
$\frac{5}{16}$	20 3 6	21 0 0

Making a deduction of $\frac{1}{16}$ inch from the thickness of the plate to allow for corrosion and contingencies, and assuming a safe working-pressure of $7\frac{1}{2}$ tons per square inch of net section of metal, the safe head of water on pipes of these thicknesses, and 30 inches in diameter, is shown by the following Table:—

Thickness of Metal in Pipe.	Safe Working-Head.	
	Riveted Pipe.	Welded Pipe.
Inch.	Feet.	Feet.
$\frac{3}{16}$	220	323
$\frac{1}{4}$	340	485
$\frac{5}{16}$	458	647

The locking-bar pipe being as strong as welded pipe, it would be possible to effect considerable economy by using $\frac{3}{16}$-inch and $\frac{1}{4}$-inch locking-bar pipes, in place, respectively, of the $\frac{1}{4}$-inch and $\frac{5}{16}$-inch riveted pipes which had been specified originally; but it was recognized in the State, when pipes of so small a thickness as $\frac{3}{16}$ inch were included in those to be tendered for, that great care would be required in handling them, in order to prevent damage; and one result of the favourably low tenders was that a minimum thickness of $\frac{1}{4}$ inch was provided throughout, thus greatly increasing the probable life of the main in the very portions where the soil is worst, and the variations in temperature greatest. Moreover, by having one thickness and one diameter throughout, the contractors were induced to make a further reduction of 5*s*. per pipe, so that the whole length of main was laid with pipes 30 inches in diameter, thus effecting some saving in the capital cost of the pumps, as well as in the cost of pumping.

Summing up the position, the results of adopting locking-bar pipes and a uniform diameter throughout are these:—The section of the ground traversed by the pipe-line is such that, considered purely from the point of view of obtaining minimum pressures on the main throughout, it would be advisable to vary the diameters and thus use up superfluous head; but the variation of pressure with a uniform diameter could not be large if the pumping-stations were suitably located, and this slight disadvantage was considered to be more than counterbalanced by the reduction in the cost of the pipes and the other advantages attending a uniform, and to some extent larger, main. Moreover, the substitution of locking-bar for welded pipe effected a saving of no less than 50 per cent. of the cost of the latter; and, although, as compared with the riveted pipes tendered for, the locking-bar pipes eventually provided cost $11\frac{1}{2}$ per cent. more, on the other hand, the latter were considered superior in several ways. Their frictional resistance, according to older accepted formulas, was less in the ratio of $2 \cdot 5 : 3 \cdot 1$, a difference of 25 per cent.; and the probable damage in handling $\frac{1}{4}$-inch in lieu of $\frac{3}{16}$-inch plate pipes would be less, and the probable life of the pipes would be much longer: for the actual thickness required for safe working being about as 2 of locking-bar to 3 of riveted pipe, the substitution of $\frac{1}{4}$-inch plate locking-bar pipe for $\frac{3}{16}$-inch riveted pipe meant a provision of $\frac{5}{24}$ inch of plate in place of $\frac{1}{16}$ inch for corrosion and damage; and the substitution of $\frac{1}{4}$-inch locking-bar pipe for $\frac{1}{4}$-inch riveted meant a provision of $\frac{7}{48}$ inch for corrosion in place of $\frac{1}{16}$ inch in the case of the riveted pipe, a difference, therefore, of 133 to 233 per cent.

As the adoption of locking-bar pipes obviated the serious and continuous loss of water which was to be anticipated from a pipe having multitudinous rivet-holes, the question was considered whether the soils in which the pipe would have to be laid would tend to shorten its life, and if so, to what extent. As already mentioned, the natural water obtainable on the goldfields is highly mineralized; moreover, it often contains free acids. Therefore thin unprotected pipes in contact with this water could not have any lengthy life—a conclusion which experience has confirmed; but careful analysis of the soils along the pipe-track (Table XI.) showed that, where mining-operations did not entail distribution of such water on to the soil in which the pipes might be buried, this soil has been so much leached as to have lost many of its harmful properties, except, of course, in the salt-impregnated beds of the so-called "lakes." It was decided, therefore, that in the latter situation the pipes should be laid on trestles above ground,

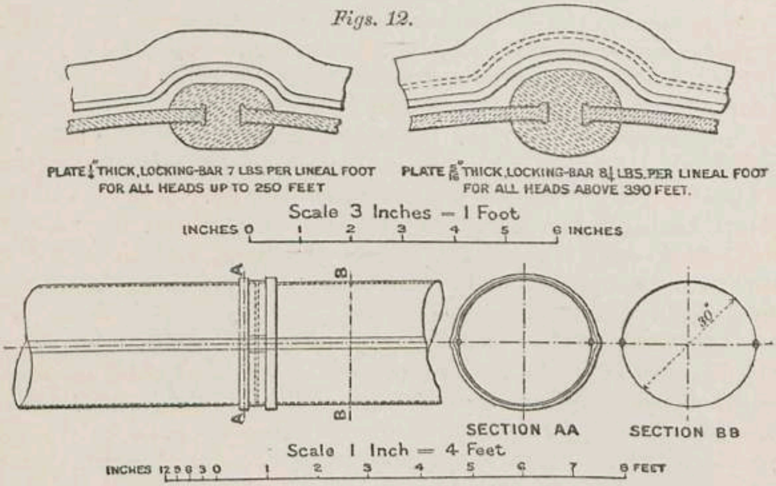
but covered with a low roof of galvanized iron; and that in the remainder of the section they should be buried, thus obviating any necessity for expansion-joints and permitting, in fact, the use of ordinary lead jointing.

The Coating.—In determining the composition of the coating to be used, wide extremes of temperature had to be allowed for. The fierce and continuous heat of the goldfields summer, when the temperature in the sun attains 150° to 170° F., is sufficient to render even block asphalt plastic. On the other hand, the frosts of winter would injuriously affect too hard a coating; and, moreover, as experiments showed, the extreme dryness of the soil at certain seasons, together with the heat, would very likely cause some loss of essential oils. As the result of a large number of tests of mixtures, made both at the pipe-works and at the head office, the coating used consisted of one part of asphalt and one part of coal-tar applied as described later, and freely sprinkled with sand while still hot and soft, to reduce the risk of the coating running when exposed in hot weather. No doubt the latter object could have been attained by more boiling, but the harder coating-mixture would have been brittle and more liable to flake off the pipes. Even the coating used ran to some extent when exposed for many days to the hot sun; but all exposure of metal, owing to this and other damage, was systematically made good just before the pipes were buried. The inside of the pipes were similarly coated—except, of course, that no sand was applied; but, as the water passing through is soft, although containing 20 grains of solids per gallon, and as vegetable acids are absent, much corrosion of the interior surface is not anticipated: and where the pipes have been emptied and opened 12 months after water started to pass continuously through them, the interior has appeared to be as clean and good as when they were first laid.

Joints.—A simple sleeve joint (*Figs. 12*) was adopted, the ring being 8 inches wide, and ½ inch larger internally than the pipe externally, to allow for slight variations in the ring, and to permit of the use of lead filling throughout. For working-heads of 320 feet and less, the section of ring used was as shown in *Fig. 13*, the weight being 126 lbs.; but for heads of more than 320 feet a stronger form was used, as shown in *Fig. 14*, the weight per ring being 160 lbs. The finished jointing has proved very effective, the loss through leakage being small. From the pipes alone, on Sections 1–5, it was found to be 343 gallons per mile per diem. From the whole length of 295 miles between the storage reservoir at Mundaring and the last pumping-station it was found to be 480 gallons

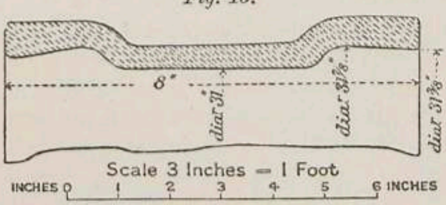
per mile per diem, over 10 months' working. This figure includes losses due to evaporation and percolation from nine pumping and

Figs. 12.



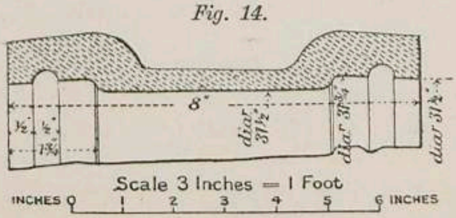
break-pressure reservoirs of contents aggregating $16\frac{1}{2}$ million gallons.

Fig. 13.



As a direct line from the Helena reservoir to Coolgardie does not deviate far from the railway already built to the gold-

Fig. 14.



fields, it was resolved that from Northam eastward the pipes should be laid parallel with the railway and at a distance of

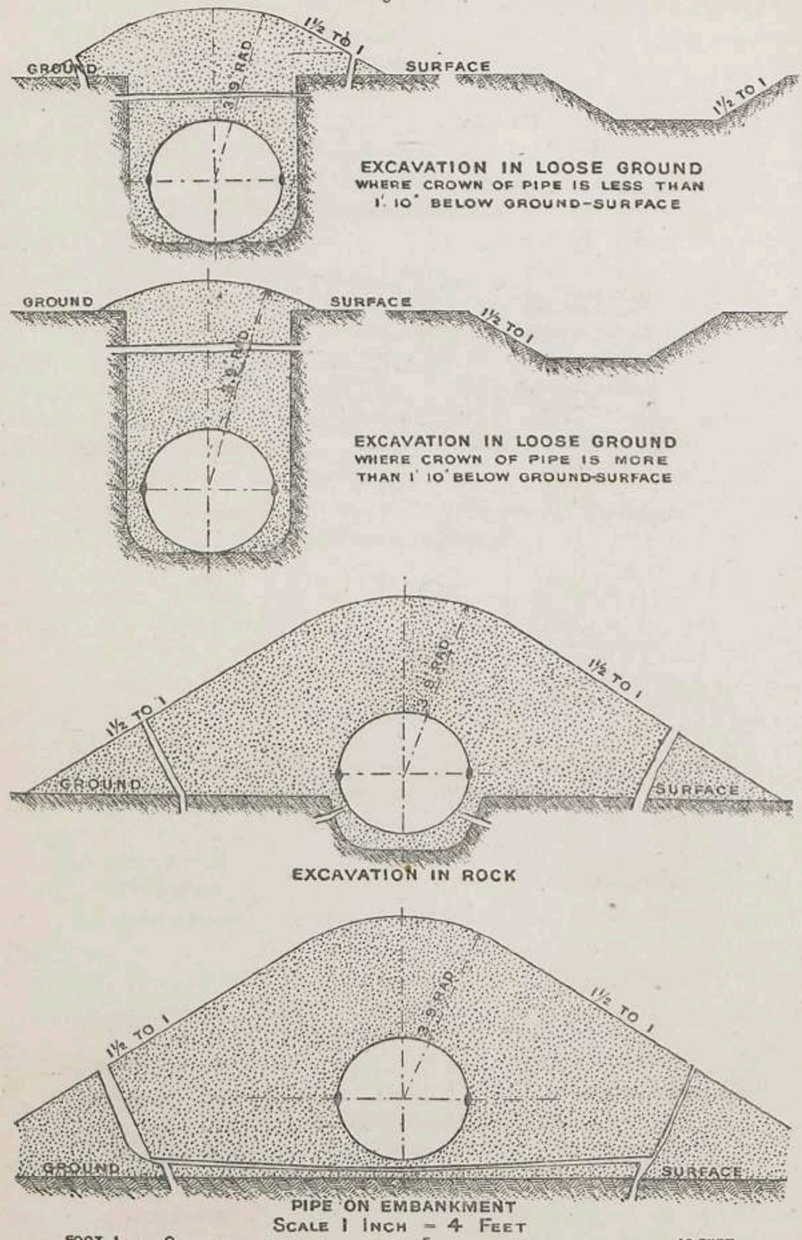
45 feet therefrom; thus gaining the great advantages of easy carriage and, subsequently, of easy supply of water to the railway: but between the weir and Northam the railway was deviated from, in order to shorten the distance, and also for the purpose of traversing higher country and thus reducing the pressure on the pipe.

Figs. 15 show the cross section of the pipe-trench and covering adopted. Where salt lakes or their beds occur, the main is carried on timber trestles, the pipes being surrounded by an insulation of saw-dust, which is kept in place by galvanised corrugated iron. This arrangement has been quite successful, no movements at the joints having taken place. Across the Avon River the pipe is duplicated, sunk beneath the bed of the river, and embedded in concrete. At railway- and road-crossings, the pipe is also protected by a shield of concrete.

At intervals of about 5 miles, stop-valves (*Figs. 16*) are inserted, the diameter of the main being reduced by cast-steel reducing-pieces to 21 inches. Where long rising gradients occur, reflux-valves are placed, the pipe being similarly reduced. Scour-valves are provided where required at both stop-valves and reflux-valves. The stop-valves are actuated by slow-motion gearing, and, on sections where the water-hammer was likely to be considerable, small by-passes were introduced, and so controlled that the water was brought to rest very slowly. Air-valves of the Glenfield pattern were placed at all summits, a nest of three being placed at the highest points, a nest of two at intermediate points, and a single valve at the lowest points. These valves are of the double type, provision being made for a large escape of air when charging the main, while air accumulating in the pipe is automatically discharged. This automatic discharge, instead of being obtained by varying the diameter of the ball, is effected by variation in the diameter of the orifice in the nipple. By this arrangement the nipple-orifices for the high points, where the pressure is light but where larger volumes of air accumulate, are of the largest diameter, and consequently afford the maximum provision for the discharge of air.

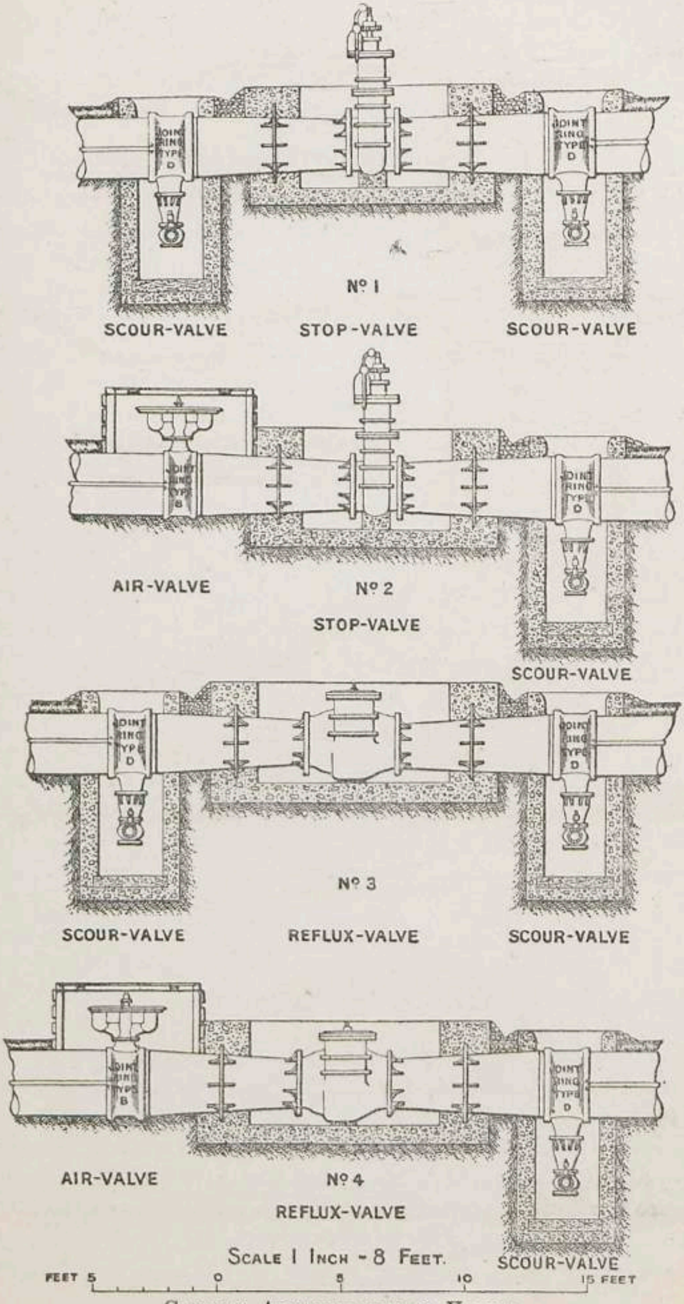
Manufacture of the Pipes.—*Figs. 12* show the section of the pipe used, the dimensions varying according to the head. A pipe consists of two plates, each of the full length of the pipe and each bent to a semicircle. The edges are burred or beaded to the shape of a dove-tail, and are inserted in the open jaws of heavy longitudinal bars, which are subsequently closed cold on to the edges of the plates, thus forming longitudinal dovetail joints. The steel used in both plates and bars was open-hearth basic steel with a specified tensile

Figs. 15.



Pipe in Cutting and in Embankment.

Figs. 16.



GENERAL ARRANGEMENTS OF VALVES.

strength at first of not less than 25 tons, or more than 29 tons, per square inch. Experience gained in the manufacture of the pipes, however, showed that steel of this quality was somewhat too hard for the bars, which, owing to the cold working, failed under test by the bursting of the jaws before the plates were ruptured. It was also found that when bars weighing $6\frac{1}{2}$ lbs. and $7\frac{3}{4}$ lbs. per lineal foot were used, respectively, with $\frac{1}{4}$-inch and $\frac{5}{16}$-inch plates, the bars failed before developing the full strength of the plate; consequently, the respective weights of the bars were increased to 7 lbs. and $8\frac{1}{4}$ lbs. per lineal foot, the steel in the bars being of a tensile strength of between 22 tons and 26 tons per square inch. From each week's output of pipes at the works pieces were cut and tested to destruction. The results are given in Table XI.

The pipes were made in Western Australia from imported plates and bars. Of the former, one-half were brought from Germany, and the balance from America; but all the bars (and the joint-rings) were obtained from England. The plates, which were a trifle over 28 feet long by 4 feet wide, were first passed through horizontal rollers, three above and three below, for the purpose of taking out all kinks and rendering the plates perfectly straight. They were then cut square and to the exact length of 28 feet. The planing and dovetailing machine next cut them to the exact width, and then burred the edges by means of rollers to form the beading for the dovetail joint. The plates next passed through a longitudinal press wherein both edges were given the required curvature, thus avoiding any necessity for the beading or dovetail being passed between and damaged in the curving-rollers to which the plates were now brought, to be bent into semicircles in the usual way. On completion of this process most of the scale had been loosened and detached, and the plates, having been thoroughly cleansed of all remaining scale and rust, were ready to be formed into pipes. One semi-circular plate was now placed in a row of half-circular cramps, resting on seats, and a locking-bar was fitted over each edge. Another semi-circular plate was then inverted and lowered until its edges rested in the upper grooves of the locking-bars. The upper halves of the cramps were then placed over the top of the pipe and connected to the bottom halves, and the plates were brought firmly home into the grooves of the locking-bars by tightening the cramps with cotter-pins. The pipe in its encircling cramps was then conveyed to a hydraulic closing-machine capable of developing a pressure of about 1,200 tons, wherein the locking-bars were pressed on to the plates, completing the manufacture of the pipe. The whole of the operations were performed without heating plates or bars.

Each pipe, before being passed, was subjected to a hydraulic pressure of 400 lbs. per square inch. The closing of the locking-bars was so effectual that only a small percentage of the pipes were found to sweat at the bars. These were returned to the closing-machine and re-pressed, and this was found to stop the sweating effectually. About fifty pipes failed altogether in the joint under this test.

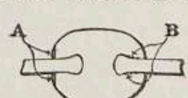
After being tested and passed, the pipes were coated. They were first heated to a temperature of 300° F. and then placed in a bath consisting of a solution of ordinary gas-tar and Trinidad asphalt, in the proportions already stated (p. 27), and kept at the boiling-point. On being lifted from the bath the pipes were allowed to drain for about a minute, and were then revolved in a machine while a jet of cold air was forced through them, for the purpose of ensuring that the coating should set in a uniform thickness. When it had cooled considerably, and just before setting, a sprinkling of sand was thrown over the outside of the pipe and gently pressed into the coating by means of rollers.

After all initial difficulties common to new methods of manufacture had been overcome, the pipes were turned out finished at the rate of 150 to 160 per diem from two factories, each of which worked two shifts of 8 hours each.

Two points required special attention in the construction and use of this pipe. The first was that the jaws of the locking-bar should be pressed well home on to the plates, no caulking of the joint being permitted at the manufactory. Unless this was very carefully done, water would enter at the ends and work along the pipe at B in *Figs. 17*, until some exit was reached. Examination of the pipe, and slight caulking at the ends before placing in trenches, disposed of such cases of opening of joints as were caused in a comparatively light pipe by handling and exposure after despatch from the manufactory. The second point also related to the necessity for closely pressing home the locking-bar, as caulking was not possible at the points marked AA: the difficulty was overcome by cutting or chipping away the portion AA.

Figs. 17.

LOCKING-BAR AND PLATES BEFORE CLOSING OF BAR



LOCKING-BAR AND PLATES AFTER CLOSING OF BAR

Conveying, Distributing and Unloading Pipes.—The whole of the pipes had to be conveyed on the trucks of a single-line railway of 3-foot 6-inch gauge. Most of them were laid alongside this line, but those which had to be taken across country where the

pipe-line deviated from the railway were conveyed from stations or sidings on specially constructed carriages. The pipes had to be distributed from the trains very quickly, so that the ordinary traffic on a fairly hard-worked railway would not be interfered with. The railway-wagons being each shorter than a pipe-length, two bogie-trucks were firmly coupled, thus giving a clear floor-length of 30 feet, and the pipes were placed thereon in three tiers. The bottom three pipes were kept in position on the trucks by means of chocks with removable gib-bolts, and three recessed bolsters, each placed across and over the bottom tier of pipes, carried the second layer, also of three pipes, which, in turn, were held in position by means of chocks and gib-bolts similar to those used for the bottom tier. In the recesses of the second layer of pipes a third tier of two pipes was placed. A truck-load therefore consisted of eight pipes, and the trucks were sent forward in trains of eleven to thirteen.

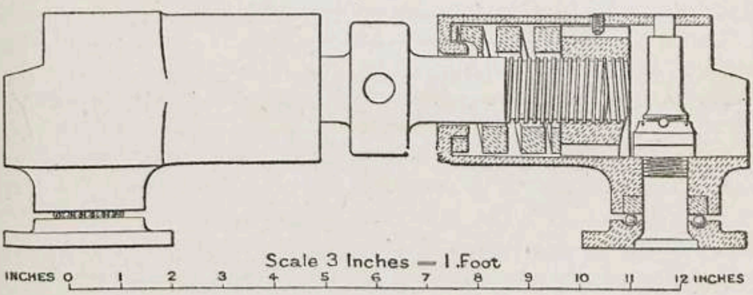
The unloading contingent of men, consisting of four gangs who, up to the arrival of the train, had been engaged on the excavation of the pipe-trench, then took charge. Each of these gangs consisted of six men, including a leading hand who controlled the gang's operations. Each gang had generally three trucks to unload, and when the train consisted of an odd number of trucks, the extra truck was allotted to the gang first getting to work. The average time occupied in unloading, from the time the loaded train left the siding until it returned thereto with the empties, was about 1 hour and 20 minutes, but the unloading was frequently done in less than 1 hour. During the remainder of the day the unloading gangs were kept at work on the excavation of the pipe-trench, sections of which had been left for this purpose. This system worked admirably, there being considerable rivalry between the various unloading gangs, and the general railway-traffic was not interrupted.

Joints.—As the whole length of main is of uniform diameter, the possibility of using machinery in place of hand-caulking of the lead joint was considered at an early stage. Careful trial at headquarters of joints caulked by hand and by a machine devised by a local firm, demonstrated that, whereas the machine-made joints when subjected to hydraulic pressure attaining 400 lbs. per square inch remained quite water-tight, on the other hand, slight sweats and pin-squirts manifested themselves in the hand-made trial joints submitted to the same test. As a 30-inch pipe of $\frac{1}{4}$-inch plate springs somewhat, even under the impact of a very light blow from the caulking-hammer, it is somewhat difficult to

get hand-caulkers to finish a joint water-tight; moreover, in practice, men working in constrained positions for long hours, in manholes under the pipe-joints, cannot be expected to do uniformly good work. On the other hand, the machine caulking can be done by pressure applied uniformly on both sides of the joint-ring, and quite as uniformly on the lower as on the upper side of the pipe. Machine caulking was therefore decided on, with the good results in freedom from leakage already stated.

The caulking-machinery consisted of a portable oil-engine of the spontaneous-ignition type, built in Australia, and of about $5\frac{1}{2}$ B.HP. The underframe of the engine also carried a dynamo which was belt-driven off the engine fly-wheel. The current was transmitted through a cable $\frac{1}{4}$ mile in length, so that about $\frac{1}{2}$ mile of pipe could be caulked before moving the generating-plant to a fresh position. The cable was coiled on a drum carried on the after

Fig. 20.



part of the transport also carrying the caulking-machine, and a plug contact was used for connecting cable and motor, so as to permit of unhampered coiling and uncoiling of the cable on the drum. The caulking-machine (Figs. 18 and 19, Plate 2) was in two halves, one fitting over and the other under the joints of the main, and on the top half of the outer frame was carried the electric motor (of 2 HP.) which was belt-connected to a shaft, and by means of intermediate gearing worked the rims holding the caulking-tools (*Fig. 20*). These rims or racks were guided by small, hardened steel rollers, grooved on the outer circumferences of the racks, but plain on the inner circumference. Into jaws on the racks were slipped the caulking-tools, two in each rack, one operating on the upper half of the joint and the other on the lower half, *i.e.*, on the underside of the pipe.

The caulking-machine was mounted on a transport on which it

D 2

was carried along the top of the pipe, from joint to joint, the lower half of the machine being slung on the transport side by side with the upper half. On arrival at a fresh joint the lower half was lifted off, placed over the joint, and slid round it to the underside; the upper half was then lowered, and the two halves were fastened together, racks were clipped and the tools placed in position, the plug-connection between drum and motor was made, and the machine was started, the caulking-tools working round the pipe backwards and forwards until the lead was pressed home. The number of semi-revolutions found necessary ranged from five, where the caulking-rollers were $\frac{1}{4}$ inch thick, to seven where $\frac{5}{16}$-inch or $\frac{3}{8}$-inch rollers had to be used to meet the varying distance between the inner surface of the joint-ring and the outer surface of the pipe. On completion of the caulking these tools were replaced by knives, which cut off the fillet in the last semi-revolution, bringing the racks back to their original position, and thus permitting the machine to be dismantled and moved to the next joint. When once fairly started, the operations were carried on without hitches, and the machinery of all descriptions, including motors and dynamos, worked well, notwithstanding that it was usually working in a cloud of dust due to the proximity of the trench-filling operations.

Each installation required three men (one a mechanic) for the working and transport of the caulking-machine, one man for the engine and dynamo, and two hand-caulkers, whose special function was to caulk at the locking-bars, whose projections prevented the rollers from working right round the pipe. In addition there were charges for parts of the time of mechanics and others whose duty it was to keep the electrical and other machinery in repair. The whole immediate cost of an installation per diem amounted to £5 1*s*. 4*d*.; and as the average day's work when the initial difficulties had been overcome was thirty-one joints, the cost per joint was 3*s*. 3*d*., or 1*s*. less than hand-caulked joints were actually costing. In addition, the saving in the average size of manhole necessary was $1\frac{3}{4}$ cubic yards; and these two savings counterbalanced the whole cost, including the patent-rights of the machinery. There is no doubt that, with the experience gained, machine-caulking could be rendered cheaper than hand-caulking, especially for a circular pipe without projections; but the object in this case was to obtain uniform and certain work, and this was attained without extra cost.

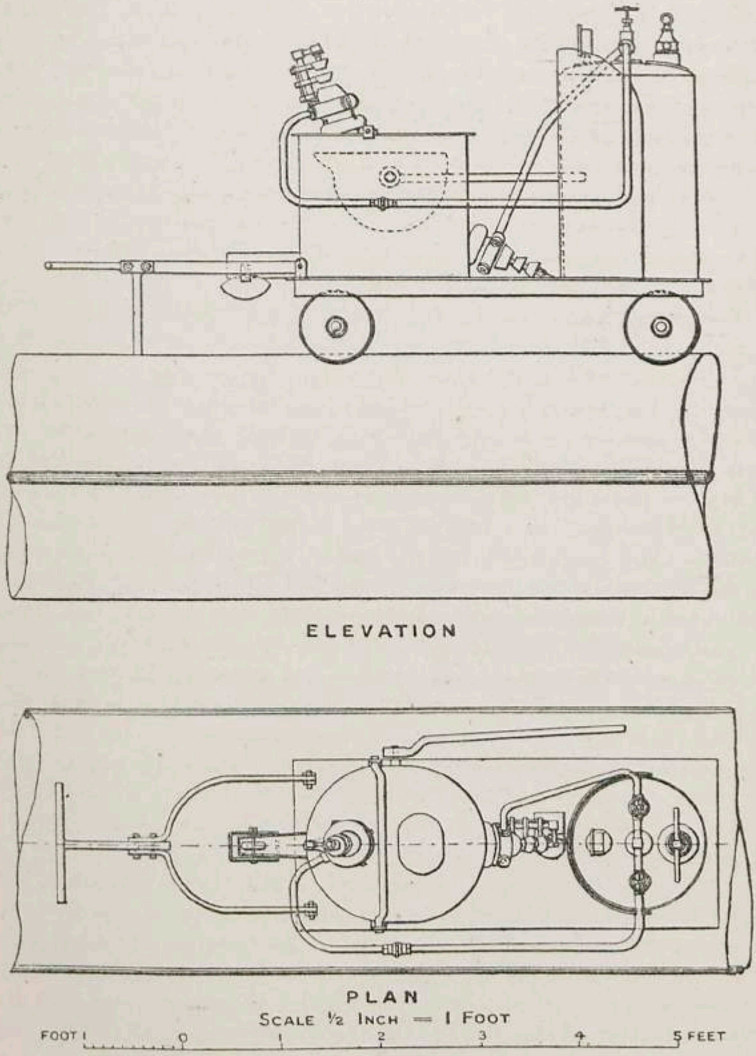
Excavation of the Trench.—The surface formation of the country traversed is very irregular. On the plains, ironstone conglomerate

predominates, but never extends continuously for more than a few chains at a stretch, being broken by bands of sand, diorite, and granite. In the timbered belts, sandy clay is the usual surface soil, but with outcrops of granite, diorite, and schist interspersed. Where at all possible, the material to be taken out was loosened by means of ploughs, each drawn by four horses harnessed in single file in the line of trench, and working to any depth required; but the bulk of the trench was taken out by manual labour, and it was necessary to use explosives on more than one-fourth of the total material removed. Where the material could be moved without the use of explosives, it was found that the most economical depth of trench, with due regard to cost of obtaining cover-material, was about 3 feet 3 inches. Where the country was harder, the trench was taken out to a less depth, the principle kept in view being that the cost of the trench, added to the cost of cover, should be a minimum. Occasionally, the contour of the ground would not admit of economical grading in combination with proper alignment for the pipes, and, in such cases, cost was subordinated to the more important consideration of easy alignment of the main. The excavation of the trench was kept well ahead of pipe-distribution, laying and jointing, but in order to provide continuous work for the gangs on these latter operations, should any hitches occur in the arrival of materials, sections of trench were left unexcavated at intervals.

Laying and Jointing Pipes and Filling in Trenches.—The work was divided into sections of about 14 miles each, to be dealt with by one caulking-installation, and when the work was completed the whole gang went forward to the next available section. When the works were brought into full swing, seven such gangs were at work on several sections; and, the class of work performed by each being identical, there was considerable rivalry between the parties. Bad work, due to haste, was prevented, however, by the appointment of an inspector on each section, who reported directly to the head office and was responsible only for the quality and not for the cost of the work, thus placing these departmental operations on the same footing as those of a contractor. The rate of progress during the last 3 months, before approaching completion caused disbanding, was, per day of 8 working-hours of seven gangs, $1\frac{2}{5}$ mile of laying, jointing, and complete filling-in of trenches. The appliances in use by each gang consisted of two pipe-lowering trestles, four skids, one pipe-expander, one lead-melter (*Figs. 21*) and retainer, and the engine and caulking-plant. The lead-running gave great trouble until the lead-melter was

devised, after which honeycombing and other similar faults were prevented.

Figs. 21.



GENERAL ARRANGEMENT OF LEAD-MELTER.

The sequence of the various operations was carefully regulated. Foremost were the men repairing the coating in the parts damaged during unloading or transportation, or where it had

become defective owing to exposure for considerable time to the intense summer heat; and in the same set were the pipe-scrapers and locking-bar chippers, who chipped or scraped off the coating at each end of a pipe for a distance of about 6 inches, to ensure good lead-running. Next came men cutting manholes, a little ahead of those laying the pipes in the trench, and following these came the ring-setters, who wedged up the joint-ring to such gauge as would give a lead joint of uniform thickness. In succession were the lead-runners, whose work was, when possible, kept at least forty or fifty joints ahead of the caulking-machine, especially in winter, as showery and cold weather affected the quality of the lead-running: thus stoppage in such weather, or defective work which had to be remedied, did not delay the caulking-operations. Following on were the hand-caulkers, who caulked the joint at the locking-bar and for a distance of about 4 inches on each side of it. The best results were obtained not by allowing the hand-caulking to finish abruptly, but by tapering up to the uncaulked portion; by this means the machine rollers were able to work by degrees well back on to the hand-caulked portion; with an abrupt finish of the hand-caulked portion the machine rollers were liable to race and cause breakages. This racing could of course have been avoided by extra care on the men's part, but at expenditure of unnecessary time, to save which would have entailed the danger of the rollers not being brought far enough along, thus leaving the joint imperfectly caulked at the junction of hand and machine work. After the hand-caulkers came the machine, and as each joint was finished the joint-inspector examined it; pipes were covered to a depth of at least 12 inches as soon as the inspector had passed a joint and it had been tarred, so that the partial filling-in was only two or three joints at most behind the machine. The completion of the filling-in and the formation of the covering bank was always 400 yards or more behind the machine.

Charging the Main.—By the 13th April, 1902, the works were sufficiently far advanced to enable pumping to be commenced with one of the engines at No. 1 station. No trouble was experienced in getting the engines under way; in fact, practically no hitch whatever occurred at any of the eight pumping-stations, and after once starting at any station the machinery was in condition to be worked, and was worked, whenever desirable. By the 22nd April the water had reached the Cunderdin Reservoir, at mile 77. Four months now elapsed before the laying and jointing of the next section was completed, and it was not till the 22nd August, 1902, that the water reached the Merredin receiving-tank at mile 140.

Some little trouble was experienced in charging this section, through the joints leaking, due mostly to the subsidence of the pipes laid across the bad ground in the bed of the Mortlock River and adjacent soft country. It was through leakage of some of the joints on this section that what may be described as "sand cuts" were first experienced. They were caused by the joint action of the escaping water and the falling sandy covering, playing together on a small portion of the outer service of the pipe. This action is somewhat similar to that of the sand-blast, and, under favourable conditions, one of the thin pipes used could be cut through in 4 to 6 hours. Fortunately, only six cases of the kind have been experienced so far. If discovered early, the placing of an encircling band on the pipe (such bands were kept in readiness) met the difficulty; but if the plate of which the pipe was made had a hole entirely or nearly cut through, a length of the main had to be emptied and the damaged pipe was replaced. To guard against occurrences of this nature, the upper halves of the lead joints were subsequently kept uncovered for some little time in country of a sandy nature, and where the main is under a head of 300 feet or more. The water reached the Coolgardie service-reservoir, at mile 328, on the 22nd December, 1902, and, finally, the Kalgoorlie service-reservoir, on the 16th January, 1903, about 8 months after the charging of the main was started. The pumping was restricted to an amount sufficient to fill about 12 to 15 miles of main per day, and, at this rate of charging, no trouble from air-pocketing was experienced, it being found that the air-valves had sufficient discharging-capacity to pass the volume of displaced and escaping air. The whole or part of the main has now been conveying water for nearly 2½ years without a burst having resulted either in the main or in the valves or other specials: the only occasions on which it has been necessary to empty any portion of the main have been when the "sand-cuts" have occurred.

IV.—The Pumping-Machinery.

Frictional Resistance of Pipes.—It was originally calculated that for a discharge of 5 million gallons per diem through 30-inch riveted pipes the frictional resistance per mile would be equivalent to a head of 4 feet, which was obtained by applying Kutter's formula with a coefficient of roughness of 0·015, a figure deduced from the measured frictional resistance of the 48-inch riveted pipe of the East Jersey (U.S.) Water Company. But the change to a much

smoother pipe allowed of a considerably reduced provision. The Commission of English engineers had proposed a frictional allowance of 2·5 feet per mile for welded pipe; but, in view of the class of water to be dealt with, this allowance was increased by 20 per cent.; and as it was further determined to increase the daily quantity to 5,600,000 gallons, the ultimate allowance was raised to 3·76 feet per mile. On completion of the works, two tests, each of 12 hours' duration, were made, one over 22 miles and the other over 12 miles of pipe, the results on reduction showing an average resistance equal to 2·25 feet per mile for a discharge of 5,000 000 gallons per diem, or 2·8 feet for 5,600,000 gallons. These results, being for clean pipes, are considerably less than the ultimate estimates; and this was foreseen, for reference to Fig. 4, Plate 1, shows that the main was laid to an even less fall than 2¼ feet per mile, in order to save unnecessary present pumping.

The total ultimate friction-head for the whole distance from the weir to the main service-reservoir at mile 307½ of the aqueduct, calculated at 3·76 feet per mile, amounts to 1,156 feet, and the natural lift to 1,290 feet; and the aggregate loss at seven pumping-stations for reservoir provision being 122 feet, the total head to be provided for is 2,568 feet: but elevated ground between pumping-stations Nos. 2 and 3 made it necessary to raise water 87 feet higher than if the slope had been gradual, thus making the total head to be pumped against 2,655 feet.

The great advisability of keeping the machinery to uniform size and pattern finally determined that the pumping-stations, eight in number, should provide for a total lift, including friction, of 2,700 feet—or 45 feet more than was absolutely necessary—namely, 450 feet at the first four stations, and 225 feet at the last four. The waste head thus amounts to a trifle less than 1¾ per cent. Moreover, in regard to the advisability of uniformity, it was further decided that the first four stations should be fitted with three groups of machinery, any two of which should be capable of performing the required work; and that the last four stations should similarly be fitted with two groups, each capable of lifting the full quantity per diem. The power necessary had thus to be the same in every group, namely, 265 effective HP., but to allow for deterioration and contingencies the pumping power contracted for was nearly 303½ HP., or about 14½ per cent. extra.

The requirements and provisions may be summed up thus:—

	HP.
Effective horse-power necessary	3,129
,, ,, provided for work . . .	3,642
,, ,, ,, as reserve . . .	2,426

Tenders for the necessary pumping-machinery were invited in April, 1899, makers being permitted to submit alternatives as in the case of the pipes. In the result a contract was entered into with Messrs. James Simpson and Co., in March, 1900, for twenty groups of machinery at an aggregate cost of £241,750, excluding spares, but including erection. A detailed description of the machinery is outside the scope of this Paper, which, however, would be incomplete without the following brief account, and results of tests.

Description of Machinery.—The pumping-plant consists throughout of almost identical sets, the only difference being that in the first four stations the pump-plungers are 15 inches in diameter, working against a specified head of 450 feet, while in the second four stations the diameter is 21 inches and the head 225 feet. The engines are horizontal, six-cylinder, high-duty, triple-expansion, surface-condensing engines of the Worthington duplex, direct-acting type, the diameters of the high-, intermediate- and low-pressure cylinders being respectively 16 inches, 25 inches and 46 inches, the normal stroke of the pump-plungers 36 inches, and the piston-speed 150 feet per minute. The pump-plungers are externally and centrally packed, and directly connected with the steam-pistons. The pump-valves are of stamped bronze. The steam-cylinders are jacketed throughout on heads and barrels with steam at boiler-pressure, and the steam is re-heated on its passage both from the high-pressure to the intermediate-pressure, and from the intermediate-pressure to the low-pressure cylinders. The re-heater tubes which draw their steam from the cylinder-jackets are placed low, thus being the means of drainage for both cylinders and jackets. The steam-distribution is controlled by Corliss valves, placed in the cylinder-heads, and the cut-off in all cylinders is adjustable by hand while the engines are running. From the air-pump the condensed steam passes through an exhaust heater placed in the exhaust steam-main to the condenser, and is delivered into an elevated feed-tank in place of the ordinary hot-well. From this tank the water gravitates to a Webster heater and oil-separator, where it is further heated by admixture with the jacket-condensation and with the exhaust from the boiler feed-pump. From the heater the feed-water is pumped by a Worthington feed-pump through the economizer back to the boilers. Steam is supplied by a nest of Babcock-Wilcox water-tube boilers, each designed to supply the necessary quantity for one engine, and having eighty-one tubes 18 feet long and 4 inches in diameter, a single drum 23 feet 7 inches long and 4 feet in diameter, with a

superheater placed between water-tubes and drum. A Green economizer is provided for each installation. The chimney-stacks are of steel, 5 feet in diameter; those at the first two stations are 130 feet high, at the third and fourth stations 100 feet, and at the last four stations 90 feet.

At six of the pumping-stations, reservoirs 15 to 20 feet deep have been provided adjacent to the machinery, to receive the discharge from the main and to furnish a store for the pumps to draw from; and in order to reduce suction lift and facilitate pumping, the centre line of the pump-plungers has been kept below the top of the reservoir by about 8 feet. At stations Nos. 1 and 3, however, special arrangements were necessary. At No. 1 the pumps, if connected directly with the main from the large storage-reservoir, would have been subjected to a head of about 100 feet when this reservoir was full; and at No. 3, where there is $\frac{3}{4}$ mile of main between the large reserve reservoir and the pumps, the latter might have suffered from an undesirable hammer. The difficulty was overcome at each place by the provision of a stand-pipe open above, from which, as from a reservoir, the pumps draw.

Figs. 22, Plate 3, show the general arrangement of the machinery at stations Nos. 1 to 4, and Figs. 23 at the remaining four. The stations are brick buildings with corrugated iron roofs. The engines and pumps rest on granite bed-stones supported by brick piers resting in turn on a concrete floor. The pump-ends are bolted down to the bed-stones, but the cylinder-ends are allowed to move freely on expansion-rollers. The greatest care was taken in the laying of the foundations, only the best available material being used; and so far there has not been the slightest perceptible movement in the foundations of any of the twenty groups of machinery. The lower floors of the engine-rooms are of concrete rendered with cement mortar, and the upper or working floors are of jarrah timber resting on steel joists. The floors of the boiler-rooms are of concrete.

Efficiency of the Machinery.—The tests provided for by the contract were three, namely, (*a*) for the duty of the whole machinery under present conditions, that is, head low owing to clean pipes and new boilers worked to full pressure; (*b*) for the duty of the engines and pumps with steam at full pressure, but the pipes throttled to obtain ultimate estimated head; and (*c*) for the capacity of the pumps with the pipes throttled and the boilers worked at 25 lbs. per square inch below present full pressure. Tests of the machinery of 12 hours' duration, at two stations to be selected by the engineer, were provided for, and the duty stipulated for was

in test (*a*) 135 million foot-lbs. for 160 lbs. of local coal worth 10,000 B.Th.U. per pound, which was taken as the equivalent of 1 cwt. of Welsh coal; in test (*b*) 135 million foot-lbs. per million British thermal units supplied to the engines and not returned in ordinary working to the boilers; and for (*c*) the full discharge with the terminal effective pressure of the low-pressure cylinders not more than 6½ lbs. per square inch, revolutions not more than 25 per minute, and piston-speed not more than 150 feet per minute.

The stations chosen for testing were Nos. 2 and 8, two groups being picked in the former and one group in the latter. Three separate preparatory tests were made to ascertain the slip of the pumps, the results being 0·6 per cent. at station No. 2, and 0·2 per cent. at station No. 8; and the respective plunger-displacements per foot of travel were, after correction for slip, 7·3645 and 14·7215 gallons. The coal in use varied slightly in quality, the calorific value per pound assigned at station No. 2 being 9916·7 B.Th.U. and at station No. 8, 10,058 B.Th.U. The values assigned to the combustibles found in the ash-pit were 11·637 B.Th.U. and 11·142 B.Th.U.

The results of the tests were that in test (*a*) the duty per 1,600,000 B.Th.U., the assumed equivalent of 1 cwt. of Welsh coal, was 144·4 million foot-lbs. at station No. 2, and 148 million foot-lbs. at station No. 8. In test (*b*) the engine-duty was nearly 142 million foot-lbs. at station No. 2 and nearly 143 million foot-lbs. at station No. 8. In test (*c*) the capacity of pumps per diem was found to be 6,093,000 gallons at station No. 2, and 6,177,000 at station No. 8. In each case, therefore, the results attained were well over those contracted for.

V.—Pumping- and Service-Reservoirs, Reticulation, etc.

The reservoirs provided are intended for three different uses, namely, to act as receiving- and suction-tanks, to regulate flow in the main, and for service purposes. There are seven suction-tanks, namely, one at each pumping-station except the first, the pumps of which draw from the storage-reservoir at Mundaring. Of the seven, all but one are concrete-lined tanks, of which that shown in Fig. 24, Plate 3, is typical; the exception is the large 10-million-gallon reservoir at mile 77, which was built some years previously for railway purposes and was taken over, as it is large enough to furnish a substantial reserve in case of accidents to the main or other works in the preceding portion of the scheme. The regulating-tanks, two in number, are also concrete-lined and are much the same in design as, although smaller

than, the receiving- and suction-tanks. That at Baker's Hill (mile 24) regulates the flow at what is (allowing for friction) the highest point on the long and irregular section between pumping-stations 2 and 3; and the tank at West Northam (mile 36) not only reduces the extreme possible pressure on the pipes in the Avon valley by 100 feet, but also permits of regulation of the flow in such manner as to keep the pressures at a minimum in regular working. The service-reservoirs are three in number, namely, one of 1 million gallons at Coolgardie, one of 2 million gallons at Kalgoorlie, and the large one at Bulla Bulling. The two smaller reservoirs are concrete-lined, and otherwise much the same as the receiving- and suction-tanks above mentioned, being also provided with by-passes so that in case of accident or necessary cleaning the working of the scheme need not be interrupted.

The main distributing-reservoir at Bulla Bulling (Figs. 25, Plate 3), which has a capacity of 12 million gallons, with an available depth of 20 feet, is rectangular in shape (Figs. 26 and 27), two of the sides having slopes of 1 to 1 for the full depth of the reservoir, while the other two sides are vertical for a water depth of 8 feet from the top and then slope to the bottom of the reservoir. The vertical portion or wall rests on a bench 6 feet wide, from the inner edge of which the sloping lining is carried down to the bottom of the reservoir. The material of the reservoir-basin consists of indurated clay, ironstone conglomerate, and bands of limestone, the whole being badly fissured and pervious to water, and liable to disintegration and to slides due to greasy backs. The Author's experience of concrete-lined reservoirs on the West Australian goldfields had been such as to show conclusively that concrete lining, even 2 feet thick, would crack when exposed to the sun; and, moreover, the cost of thick lining in a reservoir of this size would have been excessive. It was therefore determined to limit the thickness of lining to 12 inches and to provide joints in the concrete to take the inevitable movements due to expansion and contraction.

The concrete used in lining both floors and walls was composed of 5 parts of machine-broken granite, the stones being of a maximum size of 2 inches, $2\frac{1}{2}$ parts of sand, and 1 part cement; all measured by bulk. What is commonly considered the only good class of sand was not obtainable nearer than 40 miles from the work, and the cost of carriage would have been heavy; but only 1 mile away there was found a very fine sand containing 5 per cent. of clay, and 15 per cent. of very fine

powdery silica, easily movable on washing. The loam, combined

Fig. 26.

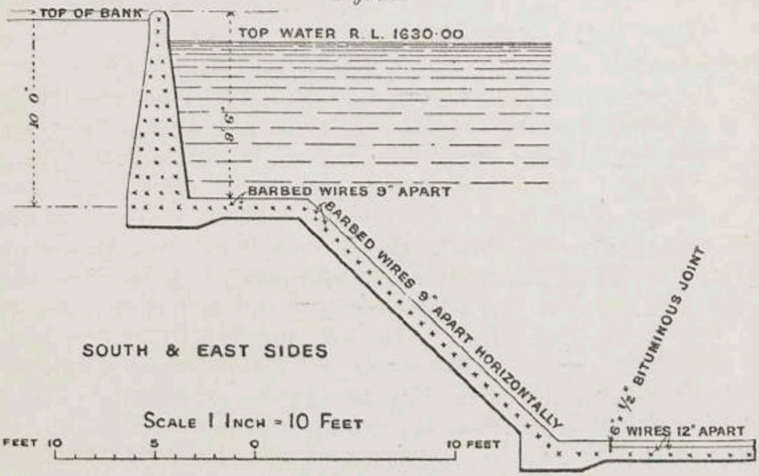
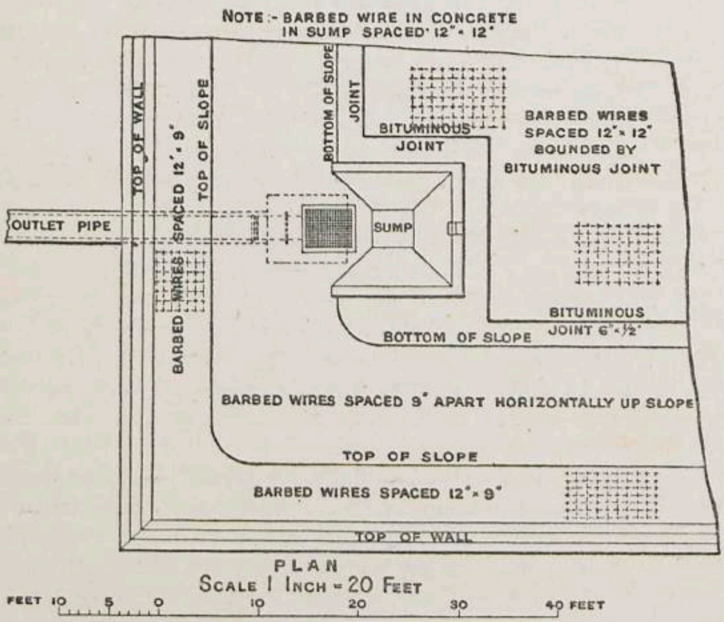


Fig. 27.



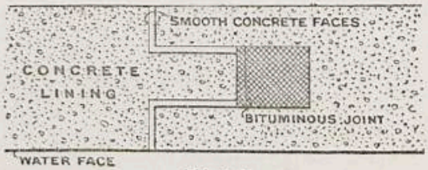
BULLA BULLING RESERVOIR: METHOD OF CONSTRUCTION.

with the extreme fineness of the sand (only 2 per cent. being

retained on a 250-mesh sieve, and $3\frac{1}{2}$ per cent. on a 400-mesh) did not at first promise good results, but the mortar tests proved very satisfactory, and in fact the briquettes made with this sand (Table XII.) proved stronger than those made with the standard sand, which was clean, coarse, and sharp; the cement used for both sets of briquettes was taken from the same cask. It is generally considered that loam or clay is always injurious to cement mortars, but the results obtained in this instance throw doubt on the point, and confirm those obtained by Professor C. E. Sherman,[1] of the Ohio State University, which showed that in practically every case the substitution of loam and clay for a corresponding quantity of sand increased the strength of the mortar.

The floor of the reservoir, 12 inches thick, was put down in two layers, the first or bottom layer being 8 inches, and the top layer 4 inches thick. In the centre of the bottom layer, a grillage of barbed wire, spaced 12 inches apart, was put in, for the purpose of adding tenacity to the concrete, and thus giving it greater power to resist cracking under contraction. The upper surface of the bottom portion of the floor was purposely left smooth, so as to allow the upper layer to slide thereon. By this arrangement, the top portion acts as a false floor, and any temperature cracks are not so liable to continue into and through the bottom portion as would be the case with the floor built in one layer. At the junction of floor and sides a bituminous joint, 6 inches deep by $\frac{1}{2}$ inch wide, is provided. The sides and walls are also reinforced with barbed wires running horizontally and placed 9 inches apart. The sides and walls were built in sections with a bituminous joint between each pair as indicated in *Fig. 28*. This arrangement effectually confined the results of contraction to the joints themselves, nearly every joint opening more or less at the faces, while the remainder of the lining remained intact. Soon after first filling, the reservoir, much of which was built in intense summer heat, was found to be leaking at the rate of $1\frac{1}{4}$ inch in vertical depth per diem; but

Fig. 28.



PLAN

[1] "Effect of Clay and Loam on Cement Mortar." *Engineering News*, 1903, vol. l. p. 443.

instead of being spread in irregular cracks all over the reservoir, the leaks were confined to the lines on which the above-mentioned joints occurred: they were easily located, and were effectually stopped by cutting out portions of the joints to a depth of 2 or 3 inches, caulking with oakum, and facing with bitumen and tar.

Reticulation.—The original scheme did not allow for any reticulation of townships for domestic purposes, or of mining-centres, it being only intended to bring the water to some high hill —for instance, Mount Burgess, a few miles north of Coolgardie —and to lay a subsidiary main thence to such situation in each township or mining-centre as the local authority should choose for its service-reservoir. Eventually, however, the complete reticulation of the townships of Kalgoorlie, Coolgardie, Boulder, and the Kalgoorlie Mining Belt, had to be undertaken as part of the main scheme, in addition to the laying of small pipes to mining-centres near Coolgardie and Kalgoorlie; but one or two of the smaller townships, namely, Northam and Southern Cross, have installed their own reticulation, purchasing water in bulk from the main scheme, and retailing it to the ratepayers.

A separate telephone-line for the works was laid down between the head office at Perth and Kalgoorlie. It is of ordinary type, with one repeating station about half-way, and was extremely useful during construction. Connection is thus secured between the head office and the pumping stations, and, by means of field-telephones, with the maintenance-gangers.

VI.—Cost of the Works.

The actual cost, including all extras, contingencies, and establishment charges, was £2,660,000, an excess of £225,000, or $9\frac{1}{4}$ per cent., on the original estimate, after deducting from the latter £65,000 for works which were allowed for therein, but were not carried out. This can hardly be considered a large excess when it is remembered that the original estimate was based on tentative data prior to survey; but as a matter of fact, almost the whole of the excess is accounted for by one item alone, namely, pumping-plant, partly due to somewhat more water having to be pumped, partly to the provision of more reserve power for accidents, and largely to enhanced cost per horse-power. Low-duty engines were originally allowed for at an estimated cost of £22 per horse-power, while the actual cost of the plant installed was nearly £48 per horse-power, including Federal customs duty, spares, etc. So

long as the consumption of water remains much below the ultimate amount allowed for, and so long as cheap local fuel (firewood) remains available, the high-duty plant will not prove as economical as the low-duty and cheaper plant would have been; but the conditions will be different when the full consumption is reached, and utilization of the more costly fuel becomes necessary.

The total expenditure of £2,660,000 was sub-divided as follows:—

	£
Storage-reservoir, including 5 miles of railway-line, land-compensation and river-training works (capacity of reservoir being 4,600 million gallons, the cost is £61 per million gallons of storage)	280,000
Service- and break-pressure reservoirs of a total capacity of 16 million gallons (£3¾ per 1,000 gallons)	60,000
Conduit 352 miles long, including all valves and specials (£5,312 per mile)	1,870,000
Pumping machinery, including erection, freight, Federal customs duty and spares (nearly £48 per horse-power)	290,000
Pumping-stations, exclusive of machinery but including the buildings, employees' quarters, suction tanks, railway-sidings, coal-staithes and stores (£23 per horse-power)	140,000
Telephone-line and other contingencies	20,000
	£2,660,000

On the death of Mr. O'Connor, in March, 1902, the Author succeeded him as Chief Engineer, when about one-half of the works had been constructed. The progress was largely governed by the necessity for testing the long lengths of main between the various pumping-stations as soon as possible, and also by the rate at which the valves and specials could be obtained from England.

In conclusion the Author wishes to express his obligation to Mr. William Coates Reynoldson for much assistance rendered. Mr. Reynoldson was the Author's principal assistant on the scheme, and is now in charge of the works as Engineer to the Trust which was constituted by an Act of the West Australian Parliament for maintenance and management of the works.

The Paper is accompanied by ninety-one sheets of drawings, from which the illustrations in Plates 1, 2 and 3, and in the text, have been prepared; also by an Appendix from which the following Tables have been selected.

[APPENDIX.

APPENDIX.

TABLE I.—DISCHARGES OF CATCHMENT-BASIN OF HELENA RIVER AND OF OTHER STREAMS.

Year.	Helena River. Above Weir. Catchment 569 Square Miles.	Helena River. Between Weir and Station B. Catchment 50 Square Miles.	Helena River. Between Stations A and B. Catchment 10 Square Miles.	Canning River at Site of Proposed Reservoir. Catchment 282 Square Miles.	Spencer's Brook. Catchment 1⅞ Square Mile.
	Million Gallons.	Million Gallons.	Million Gallons.	Million Gallons.	Million Gallons.
1898	3,802	No record	No record	3,633	[2] 28½
1899	1,857	1,654	339	No record	19
1900	9,588	5,408	1,210	"	[2] 17½
1901	1,403	2,615	597	2,014	6¼
1902 [1]	306	845	418	705	¾

[1] To end of October only.

[2] Reservoir overflowed 9 September, 1898, and 28 July, 1900, and the water that went to waste is *not* included in the above figures.

TABLE II.—DISCHARGE OF STREAMS ENTERING MUNDARING RESERVOIR 1 JANUARY, 1903, TO 16 AUGUST, 1903.

Name of Streams.	Area of Catchment.	Total Catchment of Group of Quick- and Slow-shedding Catchments.	Discharge.	Total Discharge of each Group.
	Square Miles.	Square Miles.	Gallons.	Gallons.
Creek A	7·0	..	62,271,000	
Pickering Creek. . .	10·0	..	68,046,000	
Rushy Creek . . .	18·2	35·2	166,277,000	246,594,000
Darkin River . . .	260·0	..	234,924,000	
Helena above junction of Darkin	270·0	530·0	275,755,000	510,679,000
Falling direct into reservoir and on balance of catchments . . .	4·0	4·0	53,460,000	53,460,000

TABLE III.—RAINFALL AT PERTH GARDENS, MUNDARING AND YORK.

Year.	Perth Gardens.	Mundaring.	York.	Year.	Perth Gardens.	Mundaring.	York.
	Inches.	Inches.	Inches.		Inches.	Inches.	Inches.
1876	28·73	No record	No record	1889	39·96	No record	19·99
				1890	46·73	"	22·97
1877	20·48	"	13·90	1891	30·33	"	15·19
1878	39·72	"	19·80	1892	31·23	"	13·66
1879	41·34	"	12·50	1893	40·12	49·75	23·31
1880	31·79	"	18·57	1894	23·72	29·46	10·99
1881	24·78	"	14·85	1895	33·01	44·24	16·18
1882	35·68	"	19·48	1896	31·50	37·65	15·83
1883	39·65	"	23·96	1897	27·25	35·78	13·29
1884	31·96	"	19·17	1898	32·04	43·50	16·94
1885	33·44	"	22·19	1899	31·96	37·50	16·84
1886	28·90	"	14·18	1900	36·25	45·58	21·10
1887	37·52	"	16·79	1901	35·84	35·76	13·16
1888	27·83	"	13·66	[1] 1902	26·73	26·45	10·66

[1] To end of October.

TABLE IV.—EVAPORATION AT PERTH OBSERVATORY, AND AT MUNDARING RESERVOIR.

July, 1901, to June, 1902.

1	2	3	4
		Fall of Water-Level Reservoir.	
Month.	Evaporation at Perth Observatory.	Per Month.	Per Day.
	Inches.	Inches.	Inches.
July, 1901	2·06	1·73	0·0558
August, 1901	1·75	1·47	0·0474
September, 1901	3·20	2·69	0·0896
October, 1901	5·51	4·62	0·1493
November, 1901	7·69	6·45	0·2150
December, 1901	9·47	7·95	0·2564
January, 1902	10·22	8·58	0·2767
February, 1902	8·35–35·73	7·01	0·2503
March, 1902	8·26	6·93	0·2235
April, 1902	5·21	4·37	0·1456
May, 1902	2·27	1·90	0·0613
June, 1902	1·80	1·51	0·0503
Totals and averages	65·79	55·21	..

NOTE.—The figures in column 2 were furnished by the Government Astronomer as being the loss per diem in Perth. The total of figures in column 3 for the 4 months from 1 November, 1901, to 28 February, 1902, was obtained by observation at Mundaring Weir as 2 feet 6 inches, and the figures for each month, shown in columns 3 and 4, were calculated therefrom at the proportion obtained from column 2.

TABLE V.—ANALYSES OF WATER, FEBRUARY, 1903.

Parts per 100,000.

—	Ammonia.		Oxygen absorbed in 4 Hours.	Nitrogen as Nitrates.
	Free.	Albuminoid.		
Coolgardie Reservoir	0·008	0·0375	0·228	0·016
Mundaring Reservoir: foot of dam	0·014	0·06	0·171	0·0066
Mouth of No. 1 Creek, south side .	0·012	0·064	0·15	0·012
Centre of reservoir opposite No. 1 Creek	0·012	0·072	0·135	0·011
Centre of reservoir 1,000 yards above dam	0·01	0·086	0·121	0·0092

TABLE VI.—ANALYSES OF WATER, MAY, 1903.

Locality.	Parts per 100,000.				Grains per Gallon.		Hardness Degrees.
	Free Ammonia.	Albuminoid Ammonia.	Oxygen absorbed in 4 Hours.	Nitrogen as Nitrates.	Total Solids.	Chlorine.	
*Helena Weir .	0·004	0·040	0·190	0·0145	27·49	9·94	7·0
No. 2 Tank .	0·006	0·040	0·184	0·0131	27·35	9·94	7·5
Baker's Hill .	0·012	0·026	0·171	0·0132	30·29	10·22	7·0
W. Northam .	0·012	0·026	0·148	0·0131	26·18	10·08	7·5
*Cunderdin Reservoir . .	0·014	0·025	0·166	0·0165	24·64	10·22	7·5
*No. 4 Tank .	0·012	0·026	0·129	0·0130	23·94	10·22	7·5
„ 5 „ .	0·006	0·024	0·117	0·0198	24·39	10·36	8·0
* „ 6 „ .	0·010	0·024	0·113	0·0180	25·14	10·22	8·0
„ 7 „ .	0·015	0·020	0·166	0·0264	25·99	19·36	8·0
* „ 8 „ .	0·008	0·016	0·138	0·0210	24·88	10·22	8·0
Bulla Bulling .	0·012	0·020	0·144	0·0105	23·94	10·22	8·0
*Coolgardie Reservoir . .	0·012	0·024	0·162	0·0158	23·27	10·22	8·0
*Kalgoorlie Reservoir . .	0·006	0·020	0·159	0·0165	22·96	10·36	8·0

TABLE VII.—ANALYSES OF WATER, OCTOBER, 1903.

Locality.	Parts per 100,000.				Grains per Gallon.	Hardness Degrees.
	Free Ammonia.	Albuminoid Ammonia.	Oxygen absorbed in 4 Hours.	Nitrogen as Nitrates.	Chlorine.	
Mundaring Reservoir .	0·002	0·028	0·1	0·0085	10·5	6·5
No. 2 Receiving-tank .	0·001	0·03	0·1	0·0066	11·1	7·0
Baker's Hill Regulating-tank	0·006	0·03	0·09	0·008	13·9	8·0
West Northam Regulating-tank	0·008	0·028	0·7	0·009	14·0	8·0
Cunderdin Reservoir .	0·015	0·034	0·21	0·06	10·9	6·5
No. 4 Receiving-tank .	0·002	0·03	0·08	0·0066	13·6	7·5
„ 5 „ „ .	0·003	0·034	0·23	0·005	6·3	3·5
„ 6 „ „ .	0·01	0·034	0·16	0·011	10·3	6·0
„ 7 „ „ .	0·004	0·03	0·07	0·01	14·0	7·5
„ 8 „ „ .	0·002	0·036	0·26	0·033	6·4	3·5
Bulla Bulling Reservoir	0·002	0·032	0·21	0·0066	7·5	4·0
Coolgardie Reservoir .	0·015	0·044	0·307	0·006	5·5	3·0
Kalgoorlie Reservoir .	0·018	0·03	0·3	0·016	6·0	3·0

TABLE VIII.—ANALYSES OF WATER, DECEMBER, 1903.

Locality.	Parts per 100,000.				Grains per Gallon.	Hardness Degrees.
	Free Ammonia.	Albuminoid Ammonia.	Oxygen absorbed in 4 Hours.	Nitrogen as Nitrates.	Chlorine.	
Mundaring Reservoir .	0·010	0·025	0·320	0·020	6·2	4·0
Helena River, near Byfields Weir, No. 5 .	0·003	0·009	0·176	0·007	28·41	10·0
Pickering Brook Weir, No. 2	0·003	0·008	0·032	0·0066	10·39	3·5
Rushy Creek Weir, No. 3	0·008	0·017	0·104	0·004	24·95	8·0
Darkin River Weir, No. 4	0·004	0·011	0·224	0·004	7·62	3·5
Cunderdin Reservoir .	0·005	0·018	0·233	0·017	7·39	3·5
No. 6 Receiving-tank .	0·010	0·0080	0·178	0·0017	6·20	6·0
Toorak Reservoir . .	0·008	0·025	0·167	0·02	7·62	3·5
Bulla Bulling Reservoir	0·009	0·024	0·207	0·21	7·62	3 5
Kalgoorlie Reservoir .	0·008	0·011	0·173	0·22	8·54	3·5

TABLE IX.—ANALYSES OF CEMENT.

Origin of Cement.	Composition of Cement.						
	Silica SiO_2.	Iron and Alumina Fe_2O_3 Al_2O_3.	Lime CaO.	Magnesia MgO.	Sulphuric Anhydride SO_3.	Carbonic Anhydride CO_2.	Moisture H_2O.
	Per Cent.	Per Cent.	Per Cent.	Per Cent.	Per Cent.	Per Cent.	Per Cent.
German cement .	23·61	9·21	64·27	0·94	0·96	0·41	0·48
,, ,,	23·90	9·50	63·62	0·90	1·15	0·38	0·36
English cement .	25·11	11·39	60·81	0·84	0·76	0·60	0·34
,, ,,	25·23	11·97	60·17	1·02	1·15	0·40	0·22
,, ,,	24·30	12·10	61·32	0·94	0·81	0·51	0·22
,, ,,	24·14	11·94	61·44	0·83	0·83	0·45	0·24
,, ,,	24·11	12·01	61·10	0·99	0·98	0·46	0·30
,, ,,	23·71	11·61	62·05	0·87	1·14	0·52	0·39
,, ,,	23·05	13·35	60·02	0·68	1·56	0·40	0·23
,, ,,	24·11	12·50	61·20	1·00	0·90	0·37	0·22

TABLE X.—TESTS OF TENSILE STRENGTH OF CEMENT.

Number of Days Slaked.	Tensile Strength per Square Inch.									
	Neat Cement.						3 Sand and 1 Cement.			
	7 Days.		28 Days	3 Months	6 Months	1 Year.	7 Days	28 Days.	6 Months	9 Months
	Lbs. Cold.	Lbs. Hot	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.
0	687	571	759	906	986	1,011	162	206		
6	688	792	783	..	..	..	177	207		
8	..	..	..	780	942	1,008				
9	..	..	800							
254 (*a*)	320	..	567	801	794	9 Months 790	..	..	322	323
252 (*b*)	319	..	527	676	805	847				

(*a*) and (*b*).—The slaking was effected respectively under conditions (2) and (3) outlined in the Paper (p. 19).

TABLE XI.—ANALYSES OF SOILS ALONG ROUTE OF MAIN.

Sample Number.	Place.	Reaction.	Moisture.	Loss on Ignition = Total Organic Matter.	Total Soluble Matter.	Sodium Chloride · NaCl.	Carbonic Anhydride CO_2.	Humic Acids.
			Per Cent.	Per Cent.	Per Cent.	Per Cent.	Per Cent.	Per Cent.
			Made September, 1898.					
1	Hines Hill (surface) . . .	Alkaline	4·75	5·3	0·45	0·05	0·16	0·2
2	Hines Hill (3 feet below surface)	„	13·86	15·205	0·25	0·37	0·29	0·1
3	Southern Cross (surface) . .	„	10·36	8·9	0·212	0·14	3·85	0·56
4	Southern Cross (3 feet below surface) . .	„	10·39	15·901	0·725	0·7	14·6	Nil
5	Boorabbin (surface) . . .	Acid (faint)	2·948	4·5	0·235	0·065	0·93	0·36
6	Boorabbin (3 feet below surface)	Acid (faint)	3·08	4·410	0·188	0·03	Nil	0·04
7	Coolgardie (surface) . . .	Acid	1·58	6·97	0·24	0·04	0·12	0·16
8	Coolgardie (3 feet below surface) . . .	„	2·248	6·47	0·06	0·03	0·15	0·16
9	Yellowdine (surface) . . .	„	1·78	4·36	0·55	0·07	0·153	0·43
10	Yellowdine (3 feet below surface) . . .	Alkaline	8·62	3·77	1·9	0·04	2·05	0·038
			Made December, 1898.					
11	Yellowdine Salt Lake (surface)	„	3·9	5·27	4·076	3·6	0·59	0·16
12	Yellowdine Salt Lake (3 feet below surface)	„	3·82	5·024	4·97	4·3	0·169	0·12
13	W. Cunderdin Clay Pan (surface) . . .	„	0·352	2·20	0·278	0·09	0·44	0·36
14	W. Cunderdin Clay Pan (3 feet below surface) . . .	„	5·056	7·972	0·805	0·6	0·62	0·028
15	E. Cunderdin Sand Plain (surface) . .	„	18·9	2·244	2·45	0·08	2·56	0·38
16	E. Cunderdin Sand Plain (3 feet below surface) . . .	„	12·23	7·422	2·57	0·05	0·467	0·39

NOTE.—Moisture estimated on soils as received. Other estimations made on water-free samples.

TABLE XII.—TESTS OF SPECIMEN PIECES OF LOCKING-BAR PIPES.

Number of Pieces Tested.	Thickness of Metal in Pipe.	Weight of Locking-Bar per Lineal Foot.	Number of Pieces that Failed.		Average Breaking-Stress of Plate of Pieces which Failed.	
			In the Locking-Bar	In the Plate.	In the Locking-Bar.	In the Plate.
	Inches.	Lbs.			Tons per Sq. Inch.	Tons per Sq. Inch.
124	¼	6½	90	34	19·3	26·0
130	¼	7	47	83	22·6	26·3
3	¼	7¾	..	3	..	26·8
30	5/16	8¼	9	21	23·8	25·7

TABLE XIII.—TESTS OF BRIQUETTES MADE FROM STANDARD SAND AND FROM SAND USED FOR BULLA BULLING RESERVOIR CONCRETE.

(3 Sand to 1 Cement).

—	Breaking-Stress per Square Inch.				
	7 Days.	28 Days.	3 Months.	6 Months.	1 Year.
	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.
Sand, as used in reservoir, containing 5 per cent. loam	240	336	410	466	522
Clean standard sand . .	158	257	312	377	388

Discussion.

The President moved a cordial vote of thanks to the Author for his Paper. The President.

Mr. J. Carruthers observed that the Paper was so complete that little could be added to it. The most striking point about the undertaking was the great courage of Sir John Forrest and his Government, in undertaking so unexampled a work; especially when it was considered that in Western Australia, as in nearly all the Colonies, and even nearer home, things proposed by the Government were made into political questions, and were attacked with great vigour by the Opposition. The proposal to pump water for 350 miles naturally offered an excellent opportunity for such attack, and of it full advantage had been taken. Not only had the scheme been criticized as a whole, and the advisability of carrying it out been questioned, but throughout the whole construction of the work the closest watch had been kept for any opportunity of attack upon the methods of carrying it out. When it was remembered that, in spite of all this, practically no fault could be found with the execution of the work from beginning to end, he thought there was good evidence that it had been carried out with great care and skill. The principal credit for that must be given to the Chief Engineer, the late Mr. C. Y. O'Connor. But the ablest chief could do little unless he was well supported; and the careful and skilful manner in which all the details of the work had been designed and carried out by those in the Colony reflected the utmost credit not only on Mr. O'Connor and on his successor, the Author, but on every officer of the Public Works Department of the State. Mr. O'Connor's estimates and proposals were referred to a Commission, consisting of Mr. Deacon, Professor Unwin, and Mr. Carruthers. Mr. Mephan Ferguson's invention of the locking-bar pipe was then in quite an experimental stage, as only a few short pipes had been produced by his method, and those had been made by hand. It was uncertain whether the machinery could be made for manufacturing pipes on a commercial scale as well as they were made by hand; and the Commission, while recommending the Government to approach the inventor with a view to experimenting in that direction, assumed in the meantime that riveted pipes would be used. On account of the great length of the line and the enormous number of rivets that would be Mr. Carruthers.

Mr. Carruthers. required, there was every expectation of a great deal of leakage, and the Commission recommended that the pipes should be laid on the surface, so that the leaks could be seen and caulked as they occurred. That of course necessitated the use of expansion-joints, and some experiments were made to find a good joint for the purpose. One, made of full size and thoroughly tested at Messrs. Piggott's works in Birmingham, would have been quite satisfactory; but in the meantime Mr. Ferguson had actually completed the machinery, and had made pipes on a commercial scale with entire success. The pipes proved to be excellent, and their use so largely reduced the risk of leakage that he recommended Mr. O'Connor to put the pipes underground, except across the Salt Lakes; and—whether acting on that advice or on Mr. O'Connor's own initiative—that had been done. The cost of the pumping machinery had greatly exceeded the original estimate. His original estimate was a rough calculation of the cost of ordinary low-duty pumps; but when he received instructions to prepare specifications he was informed that the fuel would be coal, and that it would cost 32*s.* per ton. That put low-duty machinery quite out of court, and the specifications were drawn with the view of getting a very high duty. About that time there was a rush of work in the mechanical world, so much so that not a ton of the 70,000 tons of steel plates required for the pipes could be obtained in England. He visited the principal pump-makers on the Continent, and found they were so busy that many of them declined to tender, as they could not undertake delivery within a reasonable time. He then went to America, and found half-a-dozen makers who were eager to secure the contract. But they entered into a combine, with the usual result of raising the price. The English makers were not influenced towards low prices by the fact that there was a combine in America, and that the German shops were all full, and their prices also rose. Only one English firm, Messrs. James Simpson and Company, would tender for the whole of the machinery: two or three other firms tendered for one-half, but not one of them would guarantee anything like the duty that Messrs. Simpson offered; and with coal at 32*s.* per ton he looked upon the high duty as being very important, and recommended the Government rather to pay more for high-duty engines than to take low-duty engines at a lower price. The duty actually obtained, 143 million foot-lbs. per 1 million B.Th.U., was so good, that he thought a more detailed account of how the tests had been made would be extremely interesting. In America, Messrs. Snow and Company, of Buffalo, had made for the Indianapolis Water Works Company a three-

crank, triple-expansion engine, which had given a duty of 150 million foot-lbs. That result had been received with a good deal of scepticism all over America, although Mr. Carruthers believed it was correct, for the tests had been made by a very able experimenter. At the same time, the result obtained by Messrs. Simpson was certainly a record for engines of the duplex class. Mr. Carruthers.

Prof. Unwin. Professor W. C. Unwin congratulated the Author on the successful completion of some very remarkable and large works. It was certainly a novelty to pump water not only to a considerable elevation, but through a pipe-line 350 miles in length. He had recently had a letter from his friend Mr. Clemens Herschel of New York, who mentioned that he believed eight Venturi meters were placed on the eight sections of pipe-line, and thought that ought to be mentioned in the Paper. It would be interesting to know whether much use was made of those meters, and whether they had proved serviceable in measuring the quantities of water. No doubt the Venturi meter was a very useful means of determining the flow through a main. When the entire scheme came before the Commission in London it was said that the whole of the soil through which the pipe-line would run was impregnated with salt, and that it was almost impossible to place a thin steel pipe under the surface soil without incurring the risk of extraordinarily rapid corrosion. The danger was thought at the time to be so considerable that with great reluctance the Commission were driven to the conclusion that it would be better to recommend the placing of the pipe-line above ground. A secondary reason for that course was that they did at that time consider that a good deal of the pipe-line would be of riveted work. From what he had seen in America he recognized that in relatively thin steel riveted pipes the occurrence of a large number of very small leaks—inconsiderable in themselves, but very troublesome in their aggregate effect—was probable, and that in such a length of main they would be a serious evil. By placing the pipes above ground, small leaks at the rivets and seams could be dealt with without digging to find them and incurring considerable expense. Apparently, after the Commission had reported, the scare about salt in the soil subsided, and it was eventually rightly decided to place the pipes underground. At the same time, the Commission, he thought, did work out a plan on which an overground line of pipes would have been quite successful. In a climate like that of Australia, the question of expansion became serious for an overground line of pipes; but he thought that, after a good deal of testing and examination, they had found a perfectly suc-

Prof. Unwin. cessful form of expansion-joint. As to increasing the thickness of the pipes beyond what the Commission recommended, it should be borne in mind that although it was quite right to do that with pipes under ground, it would not have been so necessary if the pipes had been above ground. The Commission anticipated that there would be means of absolutely preventing any external corrosion of overground pipes. On the question of corrosion he might mention that thin steel pipes made with the locking-bar were being used on the gas-mains for the Mond gas-plant at Dudley Port, where an expedient had been adopted which seemed to be exceedingly good. The pipe was first dipped in asphalt; before the asphalt was quite dry a strip of canvas was wrapped over the whole length of the pipe; and then the pipe was dipped a second time. It was not expensive, and it was an excellent way of securing an adequate permanent protective covering to the pipe, not likely to be chipped, or broken, or cracked. When the Committee sat, a few specimens of the locking-bar pipes made by hand were before them, some of which were tested for strength; and four out of five of the pipes broke in the solid plate and not at the joint. The Committee spoke exceedingly well of them in their Report, but there was no evidence at the time that their manufacture on a large scale was possible. He was very glad that through the skill of Mr. Ferguson the difficulties of manufacture had been quite overcome. The pipes were manufactured with the greatest ease and were exceedingly satisfactory. They were smoother for the purpose than riveted pipes, and much stronger. As far as strength went, they were as good as welded pipes; and in fact there were some difficulties about welded pipes which were altogether avoided with locking-bar pipes. With regard to the use of Kutter's formula with a coefficient of roughness of 0·015 in determining the resistance of the mains, in the first place, for many years past there had been a strong tendency among some engineers to have recourse to Kutter's formula, which was purely empirical and very complicated. It had no satisfactory basis, even for the case of rivers and canals, for which it had been devised. It was now known that to a certain extent the form of the equation had been determined in order that it might be made to fit certain experiments on the very large stream of the Mississippi, with very low velocities, which would not fit in well with any other formula. But it was also now known that there was strong reason to doubt the accuracy of those experiments on the Mississippi; and therefore the one thing that determined the form of the equation was unsatisfactory. Even in the case of canals and

rivers, the testing of the formula, in comparison with others derived from river- and canal-measurements in Germany, had not shown that it had any particular advantage over much simpler formulas. Looking to the fact that in using it a coefficient of roughness had always to be assumed, which had a great influence over the results given by the formula, it appeared to him to have no advantage over simpler equations in which there were various coefficients which must be selected—coefficients which did not vary so much, and in regard to which there were much better data. Again, the coefficient 0·015 had been taken from the experiments of Mr. Herschel on a 48-inch main in the State of New Jersey. He believed those experiments had been made, with the greatest possible care, with Venturi meters which he entirely trusted; but the results were so anomalous that he did not think they formed a good basis for proceeding in any question of that kind. Mr. Herschel laid the blame on the theory of hydraulics; but it seemed to Professor Unwin that it was not the theory of hydraulics which was at fault when, in the same main, at no great intervals of time, the flow proved to be widely different. On going into the question in America he had come to the conclusion that there must be in that main some cause of variation of the flow which was not properly understood. Mr. Herschel attributed it largely to increasing roughness of the main with lapse of time; but Professor Unwin could not conceive such increase of roughness, in the short period covered by those experiments, as would account for the variations of flow actually found. His own conviction was that there must be at times obstruction in the main, either from accumulation of air, or from growths or deposits which had not been discovered. Prof. Unwin.

Dr. George F. Deacon remarked that, as a member of the British Engineering Commission appointed to report upon the subject in January, 1897, he had some knowledge of the remarkable work described in the Paper. The position of the reservoir was exactly the reverse of that of all reservoirs in England, and of most reservoirs in other parts of the world. While it was an impounding-reservoir, it was situated at the bottom of the distributing system instead of at the top. The catchment-area to that reservoir was no less than 364,160 acres. The Table on p. 10, taken in conjunction with Table III. of the Appendix, was astonishing to the English hydraulic engineer, as would be shown by a simple comparison with the basin of the Thames. Taking the minimum year—because that was the year with which hydraulic engineers were most concerned—the Thames rainfall was 21 inches and the Dr. Deacon.

Dr. Deacon. Helena River rainfall 19·3 inches, while the discharge in the former case was 4·37 inches and in the latter case 0·38 inch. In other words, the Thames area discharged in the minimum year 20 per cent. of its rainfall and the Coolgardie area 0·2 per cent. That was certainly a striking difference; and the hydraulic engineer accustomed to discharges in temperate climates had much to learn before he attempted to deal with discharges in such a region as Coolgardie. He failed to understand the observation that it had not been considered advisable to raise the reservoir-wall because the diagram of capacities (*Fig.* 5) showed that the limit of economy of the site had already been reached. It seemed to him that the diagram of capacity showed precisely the opposite. If the curve were extended to show another 10 feet depth of water, the reservoir would apparently contain 40 per cent. more water. The remark might have been intended to refer to the drainage-area and to the difficulties of obtaining more water, and in this case it might have been true; but as they stood the words were open to misapprehension. The bacterial action going on in iron pipes was a subject in which he had taken considerable interest. It was suggested in the Paper that the anaerobic action in the storage-reservoir at Coolgardie had produced a marked improvement on the quality of the water. He was unable to speak from experience of an aqueduct 350 miles in length, but in the case of an aqueduct 60 or 70 miles long he could say that it was not the anaerobic action but the aerobic action which had an important effect. It was a mistake to suppose that bacteria could not get enough air for all their purposes in a very long pipe. Although the aerobic microbes fell off towards the end of 70 miles, there was still enough oxygen to support great multitudes. That was quite obvious, and he hoped the Author would state whether, and to what extent, anaerobic action had been shown to occur. He would draw attention to the Author's observations upon the smooth surface for the passage of the overflow water down the face of the dam, and to the fact that the water clung perfectly to the whole wall-face while descending. The idea of the Paper seemed to be that it was desirable, when the water had once passed the crest of the weir, to get it to the bottom of the dam as quickly as possible. It seemed to him that precisely the opposite was the proper course—that the water ought to be kept in the air as long as possible, and be broken up and turned into spray by contact with the roughest possible masonry, in order that its harmful energy should be reduced as far as possible. The dam being 100 feet high, the velocity at the bottom of the steep smooth slope must be very considerable. In his own practice he had endeavoured to reduce

the velocity of the water as much as possible, not by means of dressed sills or ledges, but simply by leaving the stones as rough as possible. Thus, at the Vyrnwy dam, for which very large stones, weighing sometimes as much as 10 tons, had been available, the projection of the rock surface beyond the draft-line often exceeded 1 foot. There, even in the greatest flood, the water never reached the bottom at high velocity; the impression created, on the eye at any rate, was that there was no such thing as solid water at the bottom: it was perfectly white spray, which spread out from the dam over the pool below—the water-cushion as it was commonly called—the whole surface of which was disturbed by the falling of the water and entangled air for 50 or 60 feet from the base of the dam. That seemed to him to be the proper course, and he had adopted it in other cases. He did not know whether the Author had any particular reason for not showing the sections of the dam to a larger scale, but it seemed desirable that the Paper should show the exact section at the weakest place, to such a scale that it could be measured.[1] He observed that the sectional area of the base of the dam below ground was much greater than that of the dam above ground, and also that puddle had been used in the face of the dam against the foundation. Puddle implied clay, and if there was clay there must have been earth; therefore he asked why an earthen dam should not have been constructed. He had just finished an earthen dam 108 feet high, across a valley whose general dimensions were not unlike those of the Helena River site, and the cost of that earthen dam must have been very much less than the cost of the concrete dam. There were numerous examples of quite permanent earthen dams of the same kind, up to about 120 feet in height. In such a work the structure below ground—merely a puddle trench—was comparatively insignificant in cost, but in every respect as safe as the enormous thickness required for the foundations of a masonry dam. He was not decrying masonry or concrete; he had every reason to do the contrary; but he thought that a site where such a depth and width of foundation were required, where the height of the dam was not much more than 100 feet, and where earth and clay could be obtained, was not the proper place for a masonry dam. The Author gave the frictional loss of head in the pipes ultimately adopted as 2·25 feet per mile when perfectly new, and said it would not continue to be so small. Calculations, upon which the Dr. Deacon.

---

[1] The Author has kindly undertaken to furnish a large-scale section of the dam, to be placed in the library.—Sec. Inst. C.E.

Dr. Deacon. conclusions of the British Commission had been based, and which had been made principally by Professor Unwin, had given the loss of head as 2·497 feet per mile for lap-welded pipes, including a small allowance for increased rusting. A large allowance had not been thought necessary because, the pipes being of steel, it was possible to protect the interior more perfectly than if they were of cast iron, unless the fettling was much more perfect than was usual in cast-iron pipes. He was unable to follow the statements in the Paper as to "original" calculations and the use of "Kutter's formula," and in another place the reference to "older accepted formulas." The British Commission had made use only of coefficients obtained in the most recent and trustworthy experiments applied in Hagen's formula; and, with proper allowance for deterioration of the interior of the pipes, the tests given in the Paper appeared to have amply justified their conclusions. The Commission had spoken favourably of the locking-bar joint, and recommended a trial of, say, ½ mile of 30-inch pipe; but, in the absence of experience, they had thought it would not be desirable to risk adopting the new process for an aqueduct 350 miles in length. It was a most fortunate circumstance that experience had been obtained of 10 miles of 25½-inch locking-bar pipe put into use in South Australia between the date of their report and the settlement of the question for the Coolgardie main. He was exceedingly pleased to find that the joints had proved satisfactory. The locking-bar joint solved, and he hoped would continue to solve, the very serious difficulty which arose in riveted pipes from friction and leakage; as the joint, if carefully formed by proper machinery, was capable of being made absolutely water-tight. It did not appear from the Paper that any portion of the 3 per cent. loss by leakage was due to the joint; the whole of it was due either to evaporation from the reservoirs or to leakage from the lead joints. Time did not allow of discussing the question of thin steel pipes versus the ordinary cast-iron pipes. There were undoubtedly places in which thin steel pipes must be used or none at all, and the case under consideration was probably one of those. He felt compelled to strike a note of warning against the imitation of such exceedingly light concrete walls and floors as those of the smaller reservoirs. By reducing expansion and contraction, the barbed wires might retard the visible deterioration; but in such thin work they could only retard it. He would ask why, in a district where the cost of land was of no importance in relation to the cost of building, the circular form had not been preferred to the rectangular form for such reservoirs. In relation to its capacity, a properly designed

reservoir of any given material was much cheaper in the circular than in the rectangular form. In the circular form the embankments kept the walls always in compression, whereas the face of a straight embanked wall was subject, as the water rose and fell, to very irregular stresses, producing sometimes even tensile strains, which, combined with temperature strains, quickly led to disintegration, unless the walls were much thicker and stronger than the circular form demanded. In conclusion, he would point out the general features of this very remarkable scheme. The daily supply was the comparatively small quantity of 5,000,000 gallons—or 5,600,000 gallons when including all water used on the way—obtained by impounding the flow from the relatively enormous area of 364,160 acres. The whole of the water had to be pumped to an altitude of 1,290 feet at a distance of 308 miles from the reservoirs, from which point it gravitated to Coolgardie, a further $23\frac{1}{2}$ miles, and again to Kalgoorlie, making a total distance of $351\frac{1}{2}$ miles, which was more than double the distance that had ever been proposed in any rational scheme for London. Dr. Deacon.

Dr. A. W. Brightmore observed that, as the subject of masonry dams cropped up in the Paper, he wished to make a few remarks on a question which was of interest at the present time, namely, whether it might be expected that there would be tensions in vertical planes near the outer toe of a dam. Of course the actual stresses occurring in a dam could not be ascertained; but it was possible to calculate what those stresses would be on certain assumptions. It had been usual hitherto to assume that the normal stresses on a horizontal plane would vary uniformly, that was, generally speaking, when the reservoir was full they would be very nearly zero at the inner toe, and would increase to a maximum at the outer toe. The question arose, how the horizontal thrust of the water was resisted. If it was resisted by forces which were proportional to the normal stresses, then the result would be the condition of things shown in *Fig. 29* (p. 66), namely, that the reacting stresses would be parallel to the resultant force on a horizontal plane. Of course that resultant force would be the resultant of the weight of the dam and of the pressure of the water. If that assumption might be regarded as a near approximation to what actually occurred, it would be easily seen that there would be no tension in a vertical plane near the outer toe; for it would be noticed that if the small triangle near the outer toe which was bounded by a vertical plane was considered, the forces acting on it would be the stresses on that vertical plane, the reacting stresses on the base, and the weight of Dr. Brightmore.

Dr. Brightmore. that small triangle. The resultant of the reacting stresses would act somewhat above the centre of that vertical plane, but the weight of the small triangle would bring the resultant force on the vertical plane, to some point marked C in the Figure, which would be very near the centre of that plane. It had been suggested that the thrust of the water was not resisted by a stress which was proportional to the normal pressure, but was resisted in much the same way as in the section of a beam. He did not see any justification for such an assumption, but if it were true, it was evident that most of the thrust would have been resisted before coming to the outer toe, and therefore the reacting forces on the base there would be more vertical, and the effect would be that the resultant acting on a

Fig. 29.

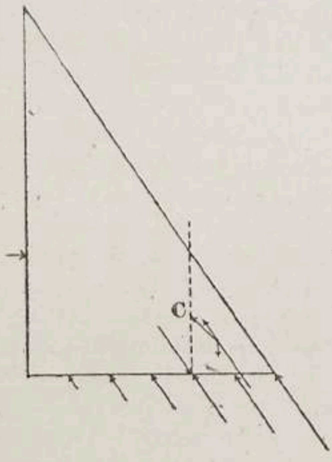
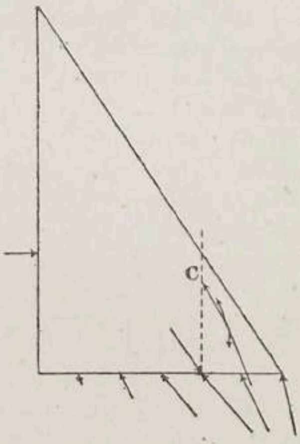


Fig. 30.



vertical plane, shown in *Fig. 30*, would be more nearly vertical; in that case some tension might be contemplated, because the resultant force on the vertical plane might quite well be outside the middle third of that vertical section. It was all a question as to what assumptions best represented the actual conditions. It appeared to him that the old assumptions should not be abandoned until it had been shown that some weakness existed in designs which had been elaborated on their basis. So far as he was aware, such weaknesses as had shown themselves in masonry structures could all be explained on the existing assumptions, and therefore he considered no departure ought to be made from those assumptions until it was proved that others were worthy of more confidence.

Mr. Maurice Fitzmaurice remarked that, considering the distance over which the pipe-line extended, and the large amount of money expended, 5 years was a short time in which to complete the work. Of course it could not have been completed in that time if great help had not been obtained from the railway which ran alongside; still, the success could only have been achieved by great forethought and organization. Considering, also, that all the pumping-plant, the pipes, the cement, etc., had had to come from England, America, or Germany, there had been more than usual good fortune in regard to delivery. Those connected with the works could point with some pride to the fact that the estimate had been so little exceeded. While, however, the engineering estimate had come out so satisfactorily, what might be called the result of the commercial estimate was not quite as good as had been expected. He understood from the Paper that the amount of water which could be delivered by the works was about 5 million gallons per day, while at the present time only about $1\frac{1}{4}$ million gallons was actually used. Such a ratio of water used to water available might be very satisfactory in a city or town with an assured future, but he did not quite see that it was so at Coolgardie. Three schemes appeared to have been considered—for 1 million, 5 million and 10 million gallons per day, respectively. If the first was worth consideration at all, it seemed rather curious that a jump should have been made to so much as 5 million gallons, and extremely fortunate that the third scheme, for 10 million gallons per day, was not adopted. Although the Author might be able to give some explanation on that point, it seemed as if there had been a considerable waste of money. Perhaps the Author could say when the 5 million gallons per day would be required. Mr. Fitzmaurice also desired to know what the present price of water was on the goldfields—whether it was based on the price given in the 5-millions estimate, which he believed was something like 6*s*. 7*d*. per thousand gallons as a maximum, or whether it had been based on the 1-million estimate. Assuming $3\frac{1}{2}$ per cent. interest on the capital of the work, and $3\frac{1}{2}$ per cent. for depreciation, or sinking-fund, making 7 per cent. altogether, which was very moderate, it would be found that the capital expenditure alone per 1,000 gallons, on the present amount of water used, came to 8*s*., without making any allowance at all for working-expenses. Like Dr. Deacon, he had rather hesitated to accept the figures for the off-flow from the area. He understood that the anticipated yield was 3 per cent. of the minimum rainfall expected, but that as a matter of fact only 0·2 per cent.

Mr. Fitzmaurice.

Mr. Fitzmaurice. had been obtained in the year 1902. Immediately afterwards the Author spoke of the question of raising the dam. Was the Author afraid that if the reservoir was filled it would not be sufficient, or was he afraid it never would be filled? Mr. Fitzmaurice had not been able to gather from the Paper which of these alternatives was feared. If the Table on p. 10, which gave the rainfall and the yield for the years 1897–1902, were extended to the years 1903 and 1904—during which period he understood the reservoir had been full—it would be of considerable interest. He could sympathize with the Author in regard to the site chosen for the dam, and quite understood what his feelings must have been when he found, after going through a considerable depth of solid granite, that he had only gone through a boulder. Mr. Fitzmaurice had had a similar experience[1]; after going through 10 feet or 12 feet of solid granite, he had come to a space about 6 inches deep partly filled with very coarse granite gravel. Of course there, as in the present case, such precautions had been taken that there was no danger of any damage to the works from that cause, the works having always been carried to a great depth below any broken ground of that sort. He desired also to know whether any other difficulty had been met with in dealing with the foundations. He noticed that it had taken about 18 months to get out the foundations for the dam, which was 800 feet long, and the Author stated that none of the masonry was commenced until all the excavation had been got out. He did not quite understand why that should be. If the trenches had been taken out to the final level as the work went on, he thought the masonry could have been commenced and carried on at the same time; and he would therefore ask why there had been delay in starting the concrete until the whole of the excavation was finished. The dam, however, was not the key to the situation, and the delay here had not done the work as a whole any harm. Judging from the amount of cement used, there appeared to be about 50,000 cubic yards of concrete in the dam, which of course was not a very large amount: reckoning $2\frac{1}{2}$ years for carrying out the work, it meant about 20,000 cubic yards per annum. He also wished to know why concrete had been used in preference to masonry. Had it been thought that concrete would make a better dam, or was it cheaper? Evidently it had not been adopted to save time. With regard to the pipe-line, he was not quite sure whether the reason for the pipes being laid underground eventually—the opposite of the recommendation of the English

[1] Minutes of Proceedings Inst. C.E., vol. clii. pp. 78, 96.

Commission—was the fact that the riveted pipes were not used, and consequently there was very little leakage, or whether the information given to the English Commission in the first place, with regard to the amount of salts in the ground, was found to be not quite correct, and that amount was much less than was anticipated. He did not quite understand from the Paper whether expansion-joints had been completely abandoned, and no allowance whatever made for expansion. If any expansion-joints had been used, it would be interesting to have a sketch of them. With regard to the rate of laying the pipes, if the rate of $1\frac{3}{5}$ mile per day had been maintained during the last 3 months, it meant that about 30 per cent. of the pipe-laying had been completed during that time. He congratulated the Author on having made very rapid progress in that final period. The service-reservoir seemed to be rather small in proportion to the large reservoir. It appeared to contain only $2\frac{1}{2}$ days' supply, which did not seem much, considering there was only one supply-pipe. It would be interesting to have the cost of the work more subdivided than it was in the Paper, so as to give some idea of the cost of the actual dam. The Author lumped together railway-line, land-compensation, river-training works, and reservoir. The statement that in practically every case the substitution of loam and clay for a corresponding quantity of sand increased the strength of mortar seemed somewhat too sweeping and indefinite, as no proportions were mentioned. No doubt in some cases a certain amount of what appeared to be clay might increase the value of mortar. Some years ago Mr. Fitzmaurice found that a large number of briquettes made with the dust from whinstone gave much better results than those made with standard sand; and he thought he remembered Dr. Deacon remarking in the Institution, some years ago, that he had found some of the material which gave the best mortar mixed with cement was a very fine material, his words being, Mr. Fitzmaurice thought, that it looked more like the sweepings from roads than anything else. About 5 or 6 years ago, Mr. Fitzmaurice made a large number of experiments on cement mortar with different proportions of clay, peat, or other impurities mixed with the sand. He found in many cases that very small quantities of these materials, say not more than $2\frac{1}{2}$ per cent., made no great difference, but 5 per cent. made a serious difference in the value of the mortar. Each case must be examined separately, and it was dangerous to make such general statements as were made by the Author with regard to the mixture of clay with cement. Mr. Fitzmaurice.

Mr. Hawksley. Mr. CHARLES HAWKSLEY, Past-President, observed that the ratio of the off-flow from the land to the rainfall was, as Dr. Deacon had pointed out, remarkable; but, of course, it had to be borne in mind that even in England the yield depended largely on the manner in which the rain fell. In two years having different total rainfalls, less water might be impounded in a reservoir in the year with the larger fall because of the rain falling comparatively lightly but frequently, so that much of the water was lost by evaporation before reaching the reservoir. It would be interesting if the Author could give some reliable information as to the amount of evaporation from the land itself in the catchment-area. That, of course, was difficult to measure: even at home no experiments had yet given thoroughly satisfactory results, and therefore such could hardly be expected from Australia. As the flow over the weir would, in times of maximum rainfall, amount to 5 feet in depth, he would ask why the weir had not been extended by carrying it farther northward along the dam. Perhaps the ground on which the water would fall was not suitable? As to the advantages of clay or loam in mortar, he remembered an instance where care had not been taken to clean the sand, and Portland cement mortar had not set after a period of 3 years, and therefore never would set. In his own practice he was always careful to have the sand used for mortar washed perfectly clean. He believed some mortar made with clay or loam had been used in the formation of the concrete on the Thirlmere works, which were described in a Paper[1] presented to the Institution some years ago. He ventured to think that the decision not to render the face of the wall with cement was wise, because such facings almost invariably peeled off in course of time, especially when subjected to damp and frost. They were applied sometimes to pathways, and there again they nearly always failed. It was much better to finish with a fine coating of concrete, and not to attempt any subsequent rendering with cement. The expansion-joints in the concrete walls of the reservoirs seemed to be well adapted to their purpose. Large masses of concrete invariably cracked somewhere; and to make provision at the place where they usually cracked, for preventing the escape of water, was a wise precaution. Having hitherto usually adhered to the cast-iron pipe, which had done good service in the past, he was unable to speak with authority

[1] G. H. Hill, "The Thirlmere Works for the Water-Supply of Manchester." Minutes of Proceedings Inst. C.E., vol. cxxvi. p. 2.

in regard to steel pipes. Of course, in countries like Australia the cost of carriage of cast pipes rendered the use of steel pipes almost a necessity, even should their life be much shorter than that of cast-iron pipes; consequently their use was justified in situations where perhaps it would not be justified in England. Also, the use of the word "stand-pipe" which occurred two or three times in the Paper was a little misleading, having regard to the fact that in England it generally meant a pipe standing above the ground to a considerable height, for the purpose of giving pressure. In the Paper the term seemed to be applied to a suction-tank. He must add his congratulations to those already given with regard to the expedition with which the work had been carried out, especially taking into consideration the difficulties in a place so far distant from the sources of most of the materials. The Author was also to be congratulated on the closeness of the cost of the work to the estimate. The excess was less than 10 per cent., which most engineers would regard as very creditable even in England, and therefore much more so in a new country like Australia, where much had yet to be learned. Mr. Hawksley.

Mr. W. B. Tripp pointed out that in his experience of the construction of waterworks in arid countries, the use of earthworks for impounding or conducting water was exposed to the risk of serious annoyance and even danger on account of the large number of burrowing animals, such as moles and rats, which constantly burrowed through the earthwork to get to the water. Sometimes these burrows were so inaccessible that they were very difficult to discover. He did not know whether that was one reason why a concrete dam, which of course was more expensive than an earthen dam, had been employed for Coolgardie; but the difficulty seemed to him to be a very practical question in connection with waterworks carried out in dry countries. Mr. Tripp.

Mr. C. T. A. Hanssen had been somewhat astonished at the statement on p. 41 as to the friction in the rising-main. The engineers had had excellent advice as to what the friction would be, and, instead of that advice being followed, 50 per cent. had been added to the frictional resistance as calculated by the highest authorities. The Author seemed to anticipate increased resistance as the result of incrustation of the pipes; but was it really necessary that pipes should silt up in the way he expected? Mr. Hanssen had had some experience at Preston with a sewage-main, 33 inches in diameter and $4\frac{1}{3}$ miles long, and he had found that with a velocity of $2\frac{1}{2}$ feet per second, which was obtained when the pumps worked at full speed, there was no Mr. Hanssen.

Mr. Hanssen. serious incrustation. In this case, in which sewage was discharged by means of Shone ejectors, stones as big as a man's two fists would traverse the main with a velocity which might be at times 3 feet, but was generally not more than 2½ feet, per second. He could quite understand that in a network of water-mains, where the velocity in one direction in any main was not very pronounced, and where there was slight oscillation to and fro, the silting might be serious; but in a case like that under discussion, where the flow was continuously in one direction through the whole of the main, as long as pumping was going on, there did not seem to be any reason why the velocity should fall below 2½ feet per second, which was generally considered a self-cleansing velocity. Although mechanical deposits could be prevented by making the velocity in the main sufficiently high, it was not certain that organic deposits could be prevented in the same way. The best way to obviate organic deposits seemed to be to exclude the light as much as possible. In the Berlin waterworks serious difficulties had arisen from the choking of the main by an organic growth, which had been found to cease when the reservoirs were covered and the light was excluded as much as possible. But in the Coolgardie scheme seven shallow reservoirs had been deliberately introduced on the pipe-line, which exposed the water as much as possible to sunlight and to air, a procedure which must encourage organic growth. Why had the lift been sub-divided in that manner? With Worthington force-pumps he did not see that there would be any mechanical difficulty in pumping the whole of the water in one lift. It would certainly cause a higher pressure on the mains—say, about 940 lbs. per square inch against the pumping-engines; but as the pressure in the hydraulic mains in London was 750 lbs. per square inch, and there was no difficulty in pumping against that pressure, he did not see why there should be any difficulty in pumping against 940 lbs. per square inch. That plan would have the advantages of not only excluding light from the water, but also of concentrating the whole of the pumping-plant in a single station—or at most in two stations—thus giving much higher efficiency, and probably reduced cost for attendance. He therefore thought the engineers had been ill-advised in subdividing the lift to such an extent, and he suggested that the high temperature of the water and its pollution with animal and vegetable organisms, which in a tropical climate would be the inevitable consequence of such an arrangement, was the reason for the low consumption of water mentioned at p. 6.

Sir Benjamin Baker.

Sir BENJAMIN BAKER, K.C.B., Past-President, considered that the Paper would be very valuable to engineers in all parts of the world, because it was clear that in South Africa and elsewhere there would be a large demand for reservoirs for irrigation and for power. He did not propose to follow the Paper through; but Dr. Brightmore's remarks seemed to indicate that he thought the question of the stresses on a dam could be solved by drawing a few lines in a diagram; and Sir Benjamin might say a few words upon that matter. When, about 12 years ago, the question of dams was under discussion in the Institution, the late Mr. C. F. Findlay, M. Inst. C.E., threw down the gauntlet by saying that assumptions had been taken by engineers as physical truths; that they thought it was sufficient to consider the stresses on a horizontal plane in a dam and to assume that the stress varied uniformly from one face to the other; but that there was no sanction to be found for such assumptions in any scientific theory which had yet been advanced, assuming the dam to be an elastic solid.[1] Many mathematicians had been working at the problem since, and during that time perhaps half a dozen masonry dams had been washed away from one cause or another; but he could not mark any great advance in the theory. It was necessary to draw a complete distinction between the mathematician's dam—that was, an elastic solid at uniform temperature—and the engineer's dam, which was quite a different thing; because there were numerous disturbing influences arising from changes of temperature, contingencies of workmanship, and other causes, which would entirely upset any reasoning based on the assumption of the dam being a perfectly elastic solid at constant temperature. In tackling the problem mathematically, however, the only way to approach it was by assuming those conditions. Then the very convenient assumption, referred to in the Paper, was made, that if the line of thrust were well within the middle third there could be no important tensile stresses on the masonry. But was that a sound mathematical theory on the assumption of an elastic solid? He thought it was not. The problem was infinitely more complicated, and he had been very pleased to see that in University College, London, after a lapse of 12 years, a serious attempt was being made by Professor Karl Pearson, an eminent mathematician and a high authority on elasticity, and by an able young demonstrator under him, Mr. L. W. Atcherley, Stud. Inst. C.E., to advance the theory of the

[1] Minutes of Proceedings Inst. C.E., vol. cxv. p. 153.

Sir Benjamin Baker.

stresses in dams.[1] About the accuracy of the results he did not care a pin. There was no pretence that the investigation was final or exhaustive, but it was the first fruits of an earnest attempt to carry the theory of an elastic solid, in the special case of a dam, a stage farther than it had been carried before. As the engineers of the Public Works Department in Egypt were interested in dams, he had transmitted the memoir to them, with the comment that he considered the investigation was deserving of every encouragement and assistance, and that although personally he could not say he agreed with the conclusions, he believed the mathematics were correct on the assumptions made, but thought that all the conditions of the problem had not been included. He had told Professor Pearson the same, and the investigations were being continued at University College. If the subject was gone into on the elastic principle, it had to be done in a thorough way. It would not do to be satisfied with merely taking the vertical stresses on horizontal planes and leaving out the stresses on other planes and the question of elastic distribution of shear. In most calculations of the kind it was assumed by engineers that if the friction at the base was sufficient to prevent the dam from sliding down-stream, that was all that was necessary to consider as regarded shearing-stresses. But Professor Pearson and other mathematicians rightly contended that if the dam was treated as an elastic solid it was necessary to consider the elastic shear, as well as the elastic compression. The first point that arose was, how was the shear distributed? If the pressure of the water was 100 tons per lineal foot of dam, and the base was 50 feet wide, was the shear distributed equally over the 50 feet, that was, at the rate of 2 tons per foot; or was it, as most mathematicians, considering the dam as a cantilever, said, distributed in a parabolic form—that was, was it nothing at the inner toe, nothing at the outer toe, and 3 tons per square foot in the centre? Professor Pearson and his colleague had worked it out on both assumptions. They said that, assuming certain other conditions, and taking the shear as uniformly distributed, there might be a tension of 3 tons per square foot at the outer toe of a typical dam in a vertical plane; but taking it as more probably parabolically distributed, then 10 tons per square foot was obtained, which was of course a

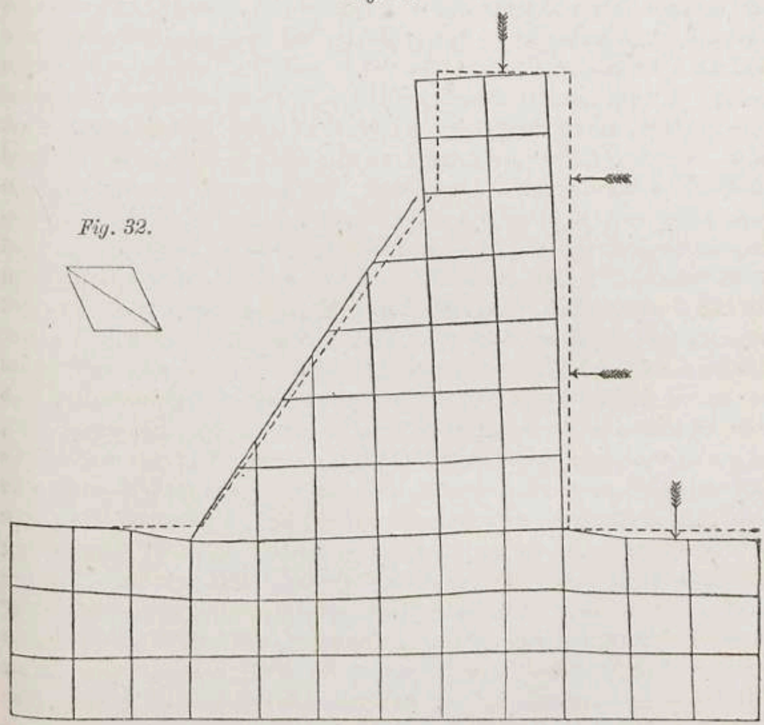
---

[1] "On Some Disregarded Points in the Stability of Masonry Dams." By L. W. Atcherley, with some assistance from Karl Pearson. [*Drapers' Company Research Memoir.*] London, 1904. For an Abstract of this memoir see Minutes of Proceedings Inst. C.E., vol. clxii.

very high tensile stress for concrete or ordinary rubble masonry. Sir Benjamin Baker. An engineer would say that he did not take either uniform or parabolic distribution, but inspected the rock upon which he had to build the dam, especially as regarded the direction and nature of the inevitable cleavage-planes, and formed his own conclusions as to what distribution of shear might be safely assumed in the particular case under consideration. But that did not invalidate the importance to an engineer of knowing what would be the distribution of stress in dealing with a dam

Fig. 31.

Fig. 32.



as an elastic solid. In dams the elastic deformations from compression would often be only comparable with the thickness of a sheet of paper in extent, and the rock on which the dam was built could not be left out of account. It was necessary to consider not only the elastic deformation of the dam, but also that of the rock. The first thing he had done on receipt of Professor Pearson's investigation was to make, in an hour, out of ordinary jelly, a model of the dam and of the rock (*Fig. 31*). He drew

Sir Benjamin Baker. lines at right angles across the transverse section, dividing it into squares, and applied pressure on the side representing the water face of the dam and also on the portion corresponding with the rock bottom of the reservoir; for it seemed to him that, although on the "middle third" theory the dam might have no tension on the up-stream side, the sinking of the rock under full-reservoir pressure might induce fairly severe tension on the masonry. The squares of the model became distorted under pressure, showing the nature of the shearing-strains, as indicated by the lines in *Fig. 32.* The first thing noticeable was that the distribution of shear where the dam met the rock was far more uniform than parabolic. In other words, if there was tension at the toe of the dam, as stated by Professor Pearson, it would be nearer 3 tons per square foot than 10 tons per square foot, so far as this rough model indicated. But the model also showed how far the strains extended into the rock, and it was probable that the elastic deformation of the dam was transmitted into the rock for a distance equal to half the height of the dam before it became undetectable. Therefore, in order to work out the complete problem, it would be necessary to take into consideration the elasticity of the rock on which the dam was built. It was a very difficult problem, especially if dealt with algebraically, without the use of models; and he could quite realize the truth of the remark made by a speaker 12 years ago, to the effect that every one who had attempted to solve it seemed, after having worked at it for a certain time, to have thrown the whole thing over and made certain assumptions to simplify matters. As he himself had remarked in the same discussion, perfect elasticity and uniform temperature might prevail on the planet Mars, but not in this world; so that engineers had to be careful in applying the results of mathematical investigation where dams were in question. In designing a dam it was possible to cover the unknown by providing a large factor of safety; but if the problem before the engineer was, "Here is an existing dam, how much can it be safely raised?" what had to be considered was the minimum factor of safety allowable. That was quite a different problem: it was like asking, with regard to a leaning tower, how much more the tower could be made to lean over: and in dealing with it great care was needed on the part of the responsible engineer—the man liable in case of accident to be called upon to produce his calculations and justify the same. Therefore, on his arrival at Assuan to determine the question of raising the dam there, in conjunction with the heads of the Public Works Depart-

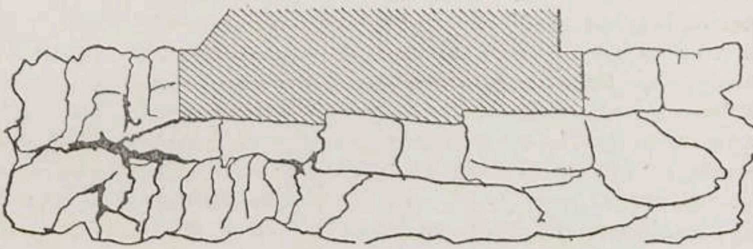
Sir Benjamin Baker.

ment he had had nearly a dozen stiff jelly models of the dam made, for the engineers to experiment with in different ways, including cutting out portions in front to represent rock scoured away by water rushing through the sluices. It was found that if rock was cut away in front of the dam the stresses on the dam were appreciably altered. After 3 or 4 hours' trial with the models, everybody had agreed with him that raising the dam was not such a simple problem as it seemed at first. He had said that for practical reasons the raising must in any event be postponed until the masonry apron below the sluices was completed, to protect the rock and support the toe of the dam; and that for the moment he would therefore not offer any opinion upon Professor Pearson's new theory, except that he was very glad the researches had been made at University College, and that he hoped the Egyptian Government would give every encouragement to further research in that direction. He thought they had done that in a handsome way: Sir William Garstin had given prominence to the researches in his Report, and had expressed the hope that Professor Pearson would be followed by other mathematicians, so that it might be possible to agree upon some complete solution of the problem of the stresses on a dam, considered as an elastic solid at uniform temperature. Of course, to assume that the rock on which a dam was founded was a homogeneous elastic solid would be very far from the truth, as Mr. Fitzmaurice's experience at Assuan had shown. With the help of Sir Archibald Geikie and some of the staff of the Geological Survey who had been in Scotland surveying the granite formations there, the matter was carefully studied by himself before the Assuan dam was commenced, and it was found that in granite formations there were invariably cleavage-planes which broke up the rock into big blocks, the spaces between which were filled with something more or less compressible—generally decomposed granite. Slow infiltration of water into cracks in the granite gradually decomposed it, the joints varying in thickness between, say, $\frac{1}{100}$ inch and more than a foot. With a solid piece of masonry in cement resting on what might be called rubble masonry set in decomposed granite, it would puzzle anyone, using any amount of mathematics, to say how the shear would be distributed over the base of the dam. Supposing that the rear part of the granite was comparatively solid and unfissured, and the front part had vertical fissures just below the surface (*Fig. 33*), fissures say $\frac{1}{4}$ inch wide, when it was considered of what order the elastic movements were—only about the thickness of a sheet or two of paper—could it reasonably be assumed that those joints

Sir Benjamin Baker.

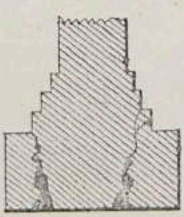
of decomposed granite were capable of resisting any effective shear? In many cases that assumption was ridiculous. That being so, then, for the toe of the dam there was no horizontal component to resist an inclined thrust. On the toe the reaction would be vertical, and its intensity would be the pressure on the rock less the weight of the toe. The dam had to be looked upon practically as an ordinary wall with footings. It must be frankly admitted that tension in masonry should be avoided

Fig. 33.



as far as possible; but the whole principle of giving a footing to a wall implied that there might be tension of some extent in any concrete or masonry structure without risk of fracture (*Fig. 34*). If the footing were made too weak it would break off, but if made strong enough it would stand. Many architects acted on the rule of thumb that the depth of the footing should be not less than the projection multiplied by the square root of the pressure on the foundations in tons per square foot; which implied a tension of 3 tons per square foot. In practice rock might be met with which was fissured and which would not afford at the toe of the dam the horizontal resistance to the inclined stress; and in that case the mortar and concrete must be strong enough to enable that part of the dam to act as a footing vertically loaded. Of course, the engineer was accustomed to providing for such contingencies. He took care in all his works, especially in dams, to test the mortar, and to make it such that the masonry or concrete had considerable tensile strength. For that reason, the fact that dams constructed on the "middle third" theory did not crack was no evidence that such theory, or any other theory of dams, was right or wrong; for in his own experience he had known rubble masonry stand a tensile stress of 20 tons per square foot

Fig. 34.



Sir Benjamin Baker.

before cracking. There was no time to elaborate the matter: he had simply tried to show that what was required from mathematicians was a complete theory on the basis of perfect elasticity in both dam and rock foundation, and he hoped some good results would be obtained at University College. When that theory had been found, engineers would have to examine the structures with which they were dealing, and if the rock they were building on was not solid rock—in the sense of being able to transmit horizontal shears of $\frac{1}{100}$ inch—they must make their dams strong enough to meet considerable tensile stresses at the toe. As he had said 12 years ago, it seemed hopeless to talk of temperatures, because whilst in dams the varying stress on the masonry from the lateral pressure of the water would be probably only about 5 to 6 tons per square foot, the modulus of elasticity and the expansibility were such that if the masonry were not free to expand, the 5 to 6 tons per square foot would be equivalent only to the stress resulting from a rise in temperature of about 6° F. The results of experience so far with thermometers buried in the masonry of dams confirmed the common-sense view that the different portions of masonry built during the year at varying temperatures settled down finally to uniform temperature in the interior of the dam, while the face-work was affected even by diurnal changes; so that internal strains existed in a dam as in a large unannealed casting. Whatever theory mathematicians might evolve, engineers would not be relieved, he thought, from the obligation to use no material for dams which would not stand, say, 50 tons per square foot in compression and 10 tons per square foot in tension without splintering; and, in some cases, concrete dams might probably with advantage be partially reinforced with steel bars.

The Chairman.

The CHAIRMAN regretted that owing to want of time the discussion could not be continued. The remarks made by Sir Benjamin Baker would be of great interest to every engineer, and valuable to the profession for many years to come.

The Author.

The AUTHOR, in reply, regarded it as a great privilege to have been able to place the Paper before the Institution, and was indebted for the kindly way in which it had been received, especially to those who, themselves responsible for large works, had drawn attention to the rapid progress made—a matter which affected him considerably. The rate of progress on the pipe-line, specially referred to by Mr. Fitzmaurice, was quite correct. The Author had undertaken that the works would be open by a certain date, and the promise had been kept. The comparatively slow construction of the weir had been due to

The Author. several causes, the most prominent being financial reasons and the advisability of even progress on the scheme generally, the weir, notwithstanding its size, forming only one-tenth of the whole work. It was difficult to deal in a few words with the political references made by Mr. Carruthers, and it was unnecessary to do so fully. It would suffice to say that Sir John Forrest's name was unquestionably and rightly one to conjure with in Western Australia, and to his Government was due the honour of inaugurating this work. There had been a great deal of needless and exasperating personal attack, and a want of recognition of difficulties and of the work accomplished; but, on the other hand, little of the opposition had been entirely factious, and, as regarded the leaders, none at all so. Moreover the Opposition of Sir John's day had certainly done their best for the scheme, and with complete success, when in power later on; at a time, too, when financial burdens were greater than with the previous Government. The work was that of many hands, and to no one person could the chief credit be apportioned; the Author had been careful to draw attention to the late Mr. O'Connor's part in it, and mention should not be omitted of one of Mr. O'Connor's assistants—Mr. T. C. Hodgson—an able engineer, whose assistance to his chief had been invaluable. The financial aspect of the question raised by Mr. Fitzmaurice was twofold; first, whether the work was too large for the purposes served, and secondly, whether as a result of the larger size the burdens were too heavy. With reference to the first point, it was easy to be wise after the event, but looking at the question from the point of view of those who had been responsible for the inauguration of the scheme, and who had had to foresee the requirements and provide for the coming large industry, and resulting population in a very dry country, it had to be remembered that the work was approved of at a time when discoveries of gold-bearing country were being made at very numerous centres (a score where mines were regularly working now could be named off-hand), the distribution of which indicated Coolgardie as being the centre of the greatest gold-bearing belt of the Commonwealth. The arid conditions of the country made very heavy demands upon the finances of the State for providing, by means of wells, bores, condensing-plants and dams, the water necessary for immediate use; while no scheme of water-supply based upon local sources was held to afford that security against prolonged drought which was an essential condition of permanent development. It would have been unwise, in the conditions which existed, and on

the data which were available, in 1897, to have erred on the side The Author.
of providing too small a scheme; and it was quite possible, having in view the immense tract of undoubtedly auriferous country served by the scheme, that the demands on further development would exceed the Author's estimate of ultimate consumption under present conditions. This estimate was 2½ million gallons, the much smaller amount noticed by Mr. Fitzmaurice being the results of the first year or so of working. Nevertheless, even during this period, the commercial aspect of the scheme was not inferior, as the following facts showed. About 3 years ago the Author submitted revised estimates of the probable cost of supply, arriving at considerably lower figures for working than the late Mr. O'Connor's, as given in the accompanying Table.

Description of Work.	Mr. O'Connor's final Figures for 5 Million Gallons sold daily.	The Author's Figures for daily quantities of		
		1 Million Gallons.	2½ Million Gallons.	5 Million Gallons.
	£	£	£	£
Interest and sinking-fund at 6 per cent. on £2,500,000	150,000	..	..	..
Interest on actual cost, including discounts on loans, etc., plus sufficient sinking-fund to redeem the loan in 20 years . .	..	194,000	194,000	194,000
Working-expenses—				
(*a*) Maintenance and general administration, excluding reticulation	61,000	35,000	35,000	35,000
(*b*) Pumping	109,000	25,000	45,000	75,000
For interest and sinking-fund per 1,000 gallons delivered at the service-reservoirs	1*s*. 8*d*.	10*s*. 8*d*.	4*s*. 3*d*.	2*s*. 1½*d*.
Ditto for working-expenses ditto .	1*s*. 10*d*.	3*s*. 11½*d*.	1*s*. 9*d*.	1*s*. 2½*d*.
Reticulation, all charges, including interest and sinking-fund .	not given	8¼*d*.	6*d*.	5*d*.
Total	..	14*s*. 8*d*.	6*s*. 6*d*.	3*s*. 9*d*.

The pipes were still new, and the cost of maintenance was therefore less than the Author's estimate; and it had been shown in actual working that for this and other reasons even his figures could be somewhat improved on. The Author made recommendations based on his figures when the Maintenance Bill was being passed, and the prices settled on for the time being were 5*s*. 6*d*. on the average at service-reservoirs (against 6*s*. 7*d*. mentioned by Mr. Fitzmaurice) and 8*s*. 4*d*. delivered at the houses. At these rates the net results of the first official year's work (the only one available so far) with the reticulation only half finished, were: revenue £104,800,

The Author. working-expenses £42,800, profit £62,000. Adding to this sum £30,000 saved to the Government railways, a total profit of £92,000 was obtained, equal to the amount of the yearly interest, leaving only sinking-fund uncovered: which was a remarkably good result for the first year's work. Turning to purely engineering matters, and beginning with the reservoir, the first point was that of discharges. He regretted that, important though the matter was, there were no records available of the evaporation from the land on the catchment, such as had been asked for by Mr. Hawksley. The small ratio of discharge referred to by Messrs. Hawksley, Deacon and Fitzmaurice was a most anxious subject for all Colonial engineers. The particular figures in this case were quite correct, as he had shown by giving those of adjoining catchments for comparison. Although anything quite so trifling as the figures for 1902 had not been met with by the Author from country chosen in other lands for catchments, yet his Indian experience had shown him that drainage-areas giving very small yields had often to be accepted; and, as recognized by the speakers, nothing but previous experience in similar country with somewhat similar distribution of rainfall could prevent waste of money on the one hand, or failure of a scheme on the other. In dealing with the relation between in-flow, consumption and loss, and the diagram of capacities, the Author, unwilling to write at unnecessary length, had rather abbreviated his remarks in the Paper, and he was glad of the opportunity afforded by Messrs. Deacon and Fitzmaurice's questions to draw further attention to the very important point of the apparently wide difference between English and Colonial practice. For instance, the late Dr. William Pole, in a lecture on "Water-Supply" delivered in the Institution 20 years ago, stated[1] that a reservoir large enough to equalize the rainfall of a series of years would be a monstrous structure. Yet it was for exactly this purpose that the Author had had to undertake a study a few years ago in India, with the intention of increasing the storage for a large town of 120,000 inhabitants; and the same question had arisen in connection with the reservoir of the Coolgardie scheme. It was natural that at any site the contents per foot of depth would be greater at the top than lower down, and it was so in the present case, as pointed out by Dr. Deacon; but, on the other hand, the loss, per unit of area, due to evaporation from broad shallow areas was greater than from areas of deeper water—a fact which used to be taken advantage of by the natives of India in the manufacture of salt by the evaporation of the brackish waters of the Sambhar

[1] "The Theory and Practice of Hydromechanics," p. 43. London, 1885.

Lake; so that, having in view the comparatively small yield from the natural catchment, it would, on working it out, be found that even though the Mundaring reservoir after enlarging would fill occasionally, small good would result from raising the dam without also increasing the catchment. As an instance, the cycle of years 1893–99 might be referred to. In the Indian case the Author had come to the conclusion that increase of storage-capacity would be money well spent; but for the Coolgardie scheme he had favoured enlargement of the catchment-area. It was impossible not to concur with Mr. Hawksley regarding the advisability generally of increasing the length of the waste-weir, and consequently reducing the depth of water over the crest. It was not advisable in the present case, owing to unsuitable ground; but the point had been fully borne in mind in the Author's preliminary studies for the yet larger dam which would ultimately be required for the water-supply of the Western Australian metropolitan area. The Author would concur with Dr. Deacon, other things being equal, if he showed preference for a clear overfall; but it was necessary in such a case that the water should shoot clear of the dam, and not jump about on the slope. It was by no means correct that the discharge reached the bottom in the form of innocuous spray, unless the depth of fall was great compared with the depth of flow over the weir-crest. One of the first works with which the Author had had anything to do was a masonry weir at Jabalpur in India, 70 feet high above bed-level at the deepest point. The dam was designed and built under the orders of Mr. J. G. H. Glass, M. Inst. C.E., and the overflow was restricted as far as possible to the flanks of the dam. The first year after the dam was completed the rainfall was exceptionally heavy, and in the 3 months during which the weir was overflowing to depths up to 2 feet 6 inches the water was usually running at sufficient velocity to drop clear of the base of the dam some 20 feet below; still some damage did occur, the joints (of lime mortar) being washed out from some of the stonework footings. Moreover there was the question of vibration, than which nothing was more injurious to concrete. In the Vyrnwy dam this perhaps did not matter. On the other hand, the Coolgardie water-supply dam, having been built homogeneous, with no surface rendering, was in little danger of scour of the profile, and the water glided down smoothly and evenly and entered the basin below without causing unnecessary disturbance. The Author failed altogether to understand Dr. Deacon's preference in the present case for an earthen dam. In the first place, it was shown in the Paper (p. 13) that the formation of a by-pass (including waste-weir and channel) would not The Author.

The Author. have been possible, unless it were cut out of the solid and sharply sloping flanks of the gorge at disproportionate expense. Again, could floods 5 feet in depth be passed over an earthen dam? Next, there was the damage and danger to such dams in countries where animal life was prolific, as pointed out by Mr. Tripp, whose remarks were in consonance with the Author's experience of maintenance and renovation in India, where not the least of the enemies of the earthen dams constructed by Indian princes in olden days were freshwater crustacea. Moreover, it was by no means a foregone conclusion than an earthen dam was the cheaper structure. The section adopted at Vyrnwy for the masonry dam built by Dr. Deacon was, the Author believed, acknowledged to be large, but taking more economical sections and comparing with those of earthen dams, and making allowances on the one hand for deep foundations and on the other hand for stripping, for puddle-tunnel, waste-weir and valve-tower, the earthen dam would not be found to be the more economical unless material were handy and labour cheap, which was not so in the present case. The question of masonry versus concrete dams raised by Mr. Fitzmaurice was one largely of comparative cost, not only of plant, materials, and rate of construction, but also of skilled and unskilled labour. A plant capable of handling large stone might have remained a dead asset, but the comparatively small stone-crushers and concrete-mixer could be counted on with certainty to be of use elsewhere. He was glad to observe that improvement had been noticed in other cases in the quality of water flowing through comparatively long pipes, although he was unable to agree with Dr. Deacon that the action was entirely aerobic rather than partly anaerobic. The arguments for and against were of a highly technical character, and would hardly be worth entering on here. While fully concurring with Sir Benjamin Baker as to the difference between a pure mathematician's assumption and an engineer's requirements in calculations for dams, the Author ventured to say that, after all, the equations of stability must remain indeterminate, and must therefore be solved by means of assumptions, as was continually necessary in numerous engineering and architectural calculations. The theory of masonry-dam calculations was sufficiently advanced, and had been sufficiently proved in practice, to ensure success, provided the designer did not cut things too fine above ground, and was careful to ensure his foundations remaining satisfactory. The latter point, as explained in the Paper, had been carefully attended to in the Coolgardie scheme; and as regarded the former, some superabundance of strength usually cost so little as compared with the whole cost of any scheme, and was moreover so necessary on account of

inevitable imperfections in materials and workmanship, that there should not be difficulty nowadays in suitably designing the above-ground portion of a dam, provided the stresses applied and the purposes to be served were carefully considered, and the extra material was correctly disposed. The Author considered, however, that Sir Benjamin Baker's remarks were directed rather to the difficulties of foundation-design, especially in circumstances where it was proposed to raise a dam which, although more than strong enough above ground, had insufficient strength below ground, because the foundations, not having been fully protected in the first instance, were considered to have subsequently become doubtful. By variation of the disposition of the superabundant material necessary at the top of every dam it was possible to deduce at will a base of greater or less width without proportionate increase or reduction in the total contents of the dam, and thus to obtain to a considerable extent the footing mentioned by Sir Benjamin Baker; but in the opinion of the Author extra width alone was insufficient where the foundation rock was comparatively weak. Extra depth was necessary, so that advantage could be taken of the resistance to vertical as well as horizontal shear of both rock and masonry below ground, and transmission therefore of stresses as through beams. If, however, this extra depth was not given, and reliance was placed on the horizontal shear alone of shallow foundations which were under threat of being curtailed by denudation down-stream, the problem was a totally different one, as well for an existing dam as for raising it. But, even so, it appeared to the Author that illustrations by means of jelly models were liable to misinterpretation, as showing more than was intended, namely, the general fact that the stresses tended to distort both dam and foundations. The stiffest jelly was not only very much more compressible than rock, but owing to viscosity the distortion was spread over a considerably greater distance than in rock, where another solution of the indeterminate equations mentioned above would be obtained by allowing for transmission of the stresses to a greater depth through the agency of unsheared beams of rock, if a sufficient factor of safety was available. In this connection, he would point out that the dam at Jabalpur already referred to, although 70 feet high above the lowest ground-surface, was built of small rough basalt blocks set in lime mortar; and it seemed unquestionable that, if instead of being regularly, evenly, and gradually distributed by means of tensile strength, the transmitted stresses were concentrated, the masonry must have failed, since lime water was not nearly so strong as cement mortar. It was, as pointed out by Sir Benjamin Baker, the right of

The Author.

The Author. engineers, who must always be greatly absorbed in practical considerations, to expect of pure mathematicians greater help in this important matter than had been accorded of late years; but it was the Author's opinion that in this, as in so many other engineering questions, theoretical calculations would not carry the matter satisfactorily to the ultimate goal, and that practical tests on a fair scale should be instituted. In these days it should not be difficult to devise satisfactory means of measuring stresses in large model dams, with suitable foundations, tested to destruction; and, considering the large issues involved, the Author hoped the Institution would take the matter up. Professor Unwin's endorsement of the use of the locking-bar pipe was noted with pleasure. Improvement of the pipe used was, however, desirable, and was possible in more than one direction. The Paper stated that the Commission did not recommend the pipe for use because it had not been proved at all in practice, and it seemed to the Author that Dr. Deacon, although fearing misapprehension, simply repeated this. The method of coating, or rather of covering, the Dudley gas-mains, referred to by Professor Unwin, had been tried on a subsidiary main of the Coolgardie water-supply, but it was too early yet to say with what results. The Author was doubtful of the utility of this protection on large mains, owing to probable hidden damage in the handling. There seemed to be no doubt that practical experience dictated concurrence with Professor Unwin in his objection to complicated formulas, and the Paper showed that Kutter's formula was not employed for the final pipe-calculations. But, this having been said, the Author could not agree that the allowance made by the English Commissioners for ultimate friction was sufficient, having in view, as the Paper stated, the class of water dealt with. This point had been missed by Dr. Deacon and Mr. Hanssen. Water drawn for the Perth water-supply from a reservoir filled from a catchment contiguous and similar to that of the Coolgardie water-supply had caused serious concretion in the mains leading to that town, notwithstanding that the water was drawn directly from the storage-reservoir and flowed faster than in the Coolgardie pipes. The Perth water held in suspension or solution not only organic matter but also iron and alumina (all, of course, in minute quantities), and the concretion produced was a totally different matter from the occasional stones Mr. Hanssen had seen transported. The extra head allowed for ultimate friction amounted to less than 10 per cent. of the whole head (frictional and gravitational), and this initial excess would not have serious effect on the economy of the pumping-engines: on the

other hand, want of sufficient allowance for ultimate frictional head must tend to produce most disappointing results later on. The Author was astonished at Mr. Hanssen's proposal to group all the pumping-plant at the weir, and to drive water thence through 350 miles of unbroken main. Had this been the only way of carrying out the work it must have remained undone, as the capital cost would have been amazing. In reply to Mr. Fitzmaurice's queries regarding the reasons for placing the main below ground, the Author would first note that apparently, according to Professor Unwin's and Mr. Carruther's remarks, the English Commission had not been in agreement as to the main reason for laying the pipes above ground. The former dwelt on the character of the soil, the latter on the loss from riveted pipes. In the Author's opinion even riveted pipes should not have been placed above ground, if this could have been reasonably avoided. The scheme was a pumping one, over undulating country, and it was always inevitable that long lengths of main would be empty from time to time; and when emptying, empty, or filling, the strains produced by unequal heating and cooling of the top and bottom of the main must have had, in riveted pipes, a serious effect in tendency towards increased leakage. It was perhaps a pity that the expansion-joints devised by the Commission had not been put to practical test, but those in the Colony who had studied the effects of the varying sun-temperatures were more than afraid that (unless placed very close together perhaps) such joints might not fulfil their purpose. The value of the water was comparatively so high, that undetected loss below ground from riveted pipes was, the Author thought, the point to be most feared, and this having been overcome by the use of another class of pipe, it was inevitable that the main should be placed below ground, the improved analysis of the soil being a solace rather than a great inducement. The extra thickness would have been as great an advantage in pipes laid above as below ground. A pipe made of $\frac{1}{4}$-inch plate was bad enough to handle, and if of $\frac{3}{16}$-inch plate the possibility of damage would have been greater, not only in handling but also when laid, unless expensively patrolled. The eight Venturi meters referred to by Professor Unwin were placed on the rising-main, one at each pumping-station, for the purpose of checking the evenness or otherwise of pumping, as well as the quantity of water sent forward. They were delicate—or perhaps more correctly susceptible—instruments, and trouble had been experienced at first in their working. When this became more successful the Author arranged for additional meters to be placed at the service-reservoirs of the various townships. The use of The Author.

The Author. the word stand-pipe, referred to by Mr. Hawksley, was possibly misleading, since this term had come to be applied to a vertical pipe used for a special purpose. The pipes referred to in the Paper were vertical and, of course, considerably above the ground; they were for the purpose of overcoming water-hammer, and not for producing pressure. The explanation given by Mr. Carruthers as to the reasons for the extra cost of the pumps over his original figures pointed more strongly, in the Author's opinion, to the doubtful expediency of installing much more expensive machinery; since if for two classes of plant the total working-costs (including interest, depreciation and working expenses) were equal, it was better, and certainly less risky, to choose the plant entailing less capital expenditure. This would have been done had the Author been in supreme charge at the time when choice was being made. There was no doubt, however, as to the high duty of the plants installed. A complete Table showing all data, also the coefficients employed and the results obtained step by step, had been placed on record in the Institution, and it need therefore only be stated that the duty-tests had been most carefully made; moreover the Author had had the whole watched and concurred in by the Chief Mechanical Engineer of the State. Prior to the test-runs, all steam- and water-gauges, thermometers, weighing-machines, etc., were tested and their errors noted. The slip of the pumps was ascertained by three initial tests on different days, the water pumped each time being from a measured 10-foot zone of the suction-tank, and amounting to 310,220 gallons. The length of stroke was measured every 5 minutes, electric signals from the tank intimating to the engine-counter reader when the top and bottom respectively of the zone had been reached. Samples of coal from various parts of the heap to be used were taken, and their calorific values were obtained independently by means of Darling, Thompson and Carpenter calorimeters; and similar care was taken with the ash-pit residue. The state of the fires, and the height of the water in the gauges at the commencement of a test, were noted, and both were brought to the same condition by the end of the test. It had not been possible, owing to the way in which the specifications were drawn, to follow in the tests exactly the lines laid down by the Institution, but had these been worked on, the duty obtained would have been still higher, for the air-pump discharges were run into a receiving-tank, and the temperature of the inflowing water was adopted as that of the condensed steam returning to the boilers in ordinary working course. There was no doubt that, as mentioned by Mr. Fitzmaurice, the service-reservoirs, specifically so termed, were small compared with what

The Author. would be provided usually; but the Author had determined that they should be sufficient at any rate to begin with, for he had provided by-passes at each, so that any one could be thrown out of working without disturbing the daily supply. Moreover, the main was of wrought-iron, and therefore not subject to the bursts which might be expected in a cast-iron main, and, in addition, the main was kept under as low a head as possible by regulating the outflowing discharge from Bakers Hill, West Northam, Bulla Bulling, and so on, instead of by means of valves at the respective receiving-reservoirs. When the demand increased largely, it would no doubt be advisable to increase correspondingly the capacity of the service-reservoirs. The advantages of the circular form in service-reservoirs, to which Dr. Deacon had drawn attention, were more or less generally acknowledged; and where deemed advisable the circular form had been adopted in the service- and suction-reservoirs of the Coolgardie water-supply, though this was not stated in the Paper. The relative advantages of thick and of thin linings, also raised by Dr. Deacon, must always be one for argument; and, personally, the Author preferred to expend £1 at once, and another £1 20 years after, rather than £1 15*s.*, or even somewhat less, to begin with. The general statement, referred to by Mr. Fitzmaurice, as to the improvement of mortar by the use of a proportion of loam in lieu of the same quantity of sand, was, of course, not based on the Author's opinion, but on the results obtained by Professor Sherman. The tests carried out for the Bulla Bulling reservoir confirmed that writer's results, and the Author would like in this connection to draw attention to the universal Indian practice of adding clay more or less burnt to pure limes. The clay was added in the presence of water, and unquestionably a stronger and more hydraulic mortar was produced than if the place of the clay had been taken by an equal quantity of sand. It was possible that similar good effects were produced by combination of clay with the free lime present in most Portland cements, but extreme care was necessary, as Mr. Hawksley had pointed out.

Correspondence.

Prof. Brown. Professor J. Campbell Brown, of Liverpool, agreed that the theoretical requirements for a good coating for iron pipes were as stated at p. 27. These desiderata, however, were not obtained by the practical means mentioned on p. 33. Not only at Coolgardie,

Prof. Brown. but also elsewhere, Trinidad asphalt was being employed instead of pitch, with gas-tar, for the purpose of coating iron pipes. But natural asphalt contained only about 14 per cent. of the matter which was really valuable for protecting iron pipes, namely pitch, the remainder being mainly calcium carbonate. This large amount of mineral matter was good for paving purposes, but for coating iron it was not only useless, but positively disadvantageous. It thickened and hardened the pitch with material which did not help to make the pitch adhere to the pipes, and which was gradually dissolved by large quantities of water containing carbonic acid or other substances which acted upon limestone. Accordingly, hard pitch from gasworks would be worth more than seven times as much as natural asphalt for mixing with coal-tar for the inside coating of water-pipes. For the outside it was not of so much importance, because there the coating might be thicker, and the pipe might be embedded in asphalt of considerable thickness with advantage.

Mr. Bruce. Mr. A. FAIRLIE BRUCE observed that the proportion of rainfall actually found to flow off the Coolgardie catchment-area was in accordance with the general experience that, with small precipitation, the percentage received in the reservoir was much less than in the case of a heavy fall, as the former generally consisted more largely of light local showers, and the water was re-evaporated or absorbed before it reached the watercourses. No doubt the small percentage collected was also due in part to the gathering-ground being so heavily timbered. Rainfall observations were liable to be very misleading, especially in a large area, unless a considerable number of rain-gauges were distributed over the area so as to give a true average result; as the amount of rainfall was greatly influenced by elevation and exposure. The experience in Bombay had been that the mean rainfall at Tansa Lake, at a height of 362 feet above sea-level, was about 108 inches, at Tulsi Lake, 370 feet high, 104 inches, and at Vehar Lake, 190 feet high, 84 inches. As measured by the rain-gauges, which were situated at or near the dams, about 50 per cent. of the total rainfall reached the Tansa, and almost 65 per cent. the Tulsi and Vehar Lakes; but as much of the gathering-grounds, especially those of the two latter, were much higher than the rain-gauges, it was believed that a deduction of 10 to 20 per cent. should be made from these figures, to obtain correct results. The inaccessibility of the surrounding hills, which were thickly clothed with jungle, prevented rain-gauges from being established on them. The Author was to be congratulated on the general design and water-tightness of the weir, though it appeared to Mr. Bruce

that the outlet-works might have been modified with advantage; for example, by making provision for the fixture of temporary external sluices, to enable the working sluices to be repaired in case of need, as had been done at the Staines reservoirs. The iron stand-pipe in the valve-tower appeared to be somewhat unnecessary. It would have been an improvement had the outer valve-chamber been transformed into a screening-well, as external screens were liable to damage and provided a very small area. The precautions taken to protect the toe of the dam from the effects of scour by water flowing over it were wise, though the impact might have been further reduced had the outer face been stepped or rock-faced. At Tansa, where of course the body of water flowing over the portion of the dam used as a waste weir was much larger, nullahs, 20 to 30 feet deep and 50 to 100 feet wide, had been cut by the water, and sooner or later protection works would become necessary; as already at one or two points the erosion had approached nearer to the foundations than was altogether desirable. The locking-bar pipe adopted seemed to have been in every respect the most satisfactory form of pipe that could have been used, as, in addition to avoiding the loss of strength due to riveting, considerable longitudinal stiffness was imparted by the bars. It would be interesting to know if the coating was proving durable, and if a perfectly smooth internal surface had been obtained. In America, where several asphaltic compositions had been employed for coating steel pipes, they were usually baked in an oven to harden and improve the surface. The Author did not mention whether yarn had been used in making the joints, or if not, what method had been adopted to prevent lead from running into the pipes. On the Glasgow waterworks, where the 48-inch pipes were jointed with collars of somewhat similar design, this was effected by means of internal rings, which were removed after the joint was run, when it was staved internally as well as externally. The reduction of diameter at the valves might have been carried even further with considerable advantage. Mr. Bruce had for some years employed 24-inch valves for 48-inch pipes, and 18-inch valves for 32-inch pipes, small valves being cheaper in first cost and much more easily manipulated, repaired, or replaced, than those of the full diameter of the pipe. Kutter's formula was quite unnecessarily complicated for calculating the discharge of pipes, that of Darcy being much simpler and more logical. Applying it to this case with a coefficient of friction of 0·00345, the results obtained by the Author would appear to be about 18 per cent. less than they ought to have been for new and well-coated pipes; and it would be interesting to know if there was anything

Mr. Bruce.

Mr. Bruce. in the roughness of the coating, or any friction due to the projection of the bar into the cross section of the pipe, which would account for this. The following were the results of some experiments made by Mr. Bruce on the discharge of large pipes :—

No.	Diameter.	Age.	Below Calculated Quantity.	Length.	Situation.
	Inches.	Years.	Per Cent.	Miles.	
1	48	New	1	1	Glasgow Waterworks.
2	48	10	7	11	Bombay ,,
3	32	42	34	13	,, ,,
4	24	24	32	16	,, ,,
5[1]	24	24	19	16	,, ,,
6	24	16	30	13	,, ,,

[1] No. 4 after being scraped.

The covering of the pipes with earth would appear to have been a wise step, though the banks would demand constant attention to prevent them from being washed or blown away. Experience in Bombay had shown that where pipes were laid below ground in dry earth, free from salt or organic impurity, they had remained for nearly 50 years practically unaffected by corrosion; whereas others laid above ground had been more or less attacked in a quarter of that period. They also required frequent caulking, owing to the expansion and contraction due to variations of temperature. Concrete lining on slopes, such as that employed in the service- and regulating-reservoirs, was usually found to give trouble, though it was possible that the system of reinforcement adopted might suffice to prevent the tendency to crack owing to slight subsidence, which generally occurred sooner or later. Had the floor been asphalted the thickness of concrete might have been safely reduced to about 6 inches, without danger of leakage.

Mr. Crowell. Mr. Foster Crowell remarked that the Paper, besides being an instructive record of excellent work, contained useful suggestions of general bearing. For example, the facts cited in regard to the faults discovered in the rock bed at the site of the Helena weir showed the meagre value of preliminary borings and trial-shafts as indicators of the conditions of any rock formation, with reference to its continuity and reliability for the purposes of a dam. This was a not infrequent experience, and had been marked by many striking examples, a prominent one being the New Croton dam, of the New York City waterworks, recently completed, where extensive and careful preliminary explorations had been made, but where great discrepancies

Mr. Crowell.

had been disclosed later by the complete excavations.[1] It might be set down almost as an axiom that the suitability of a particular rock formation for the foundation of a dam could not be known definitely until the final excavations were actually made: nevertheless, it was customary for engineers to proceed with the design, make the preliminary estimate of cost, and award the contract, as if the borings and trial-shafts were quite sufficient, trusting to luck or depending on judgment as to whether the indications were comprehensive. If afterwards serious geological faults were disclosed by the excavation, a change or abandonment of site might become necessary, or at least desirable; but usually in such cases one of two courses was adopted, either going deeper—involving increased quantities and cost, and perhaps disclosing new difficulties—or resorting to some more or lest unsatisfactory artifice with attendant risks. Guided by past experience, he had adopted some time ago a rule to assign only negative or tentative values to preliminary borings; that was to say, the absence of rock would be a positive indication of unsuitability, but no other indications would be considered positive until corroborated. Carrying out this principle in preparing plans for a masonry power-dam at St. Joseph's, N.Y., in 1903, subsequently built under his direction, he had decided to ascertain actual conditions by uncovering a large part of the proposed site down to bed-rock at the owner's expense, before completing the plans and specifications or inviting tenders. There had been special reasons for precautions here, owing to the fact that the proposed structure was to take the place of a former dam which had failed because of defective foundations. Fortunately, favourable conditions had been found; but even if it had turned out otherwise, the extra expense entailed in that contingency, regarded as insurance charges against a much greater loss, would have been amply justified, and he would advocate a similar course in all important cases. It was of interest to note the growing co-ordination of recognized engineering methods on modern works, throughout the world. It did not often happen, of course, that conditions were precisely the same in different countries; but, especially during the past few years, it had come to pass that, given like general conditions, the treatment resorted to by engineers was apt to be carried out on closely similar general lines. Even in the details, and in the technical terminology employed

[1] C. S. Gowen, "The Foundations of the New Croton Dam." Transactions American Society of Civil Engineers, vol. xliii. p. 469.

Mr. Crowell in describing results, the same was true; so that in reading and studying in America this Paper, written of work carried out in Australia—which, to Americans, was one of the uttermost parts of the earth—no glossary was required in order to arrive at a full understanding; while the work described was excellent and up-to-date, if, indeed, it was not distinctly in advance in several respects. The pipe-line was noteworthy, in respect of both the use of locking-bar pipe, and the very ingenious and effective caulking-machine; its great length and high working-pressures gave it rank among notable metal conduits; and its very fine results, as shown in the Paper, were matters of congratulation to designer and maker alike. The discharge-capacity of locking-bar pipe compared with riveted pipe of equal diameter, as shown by the tests, was a matter of great interest and importance, and it was to be hoped that the Author would supplement these tests from time to time with others, to determine the effect of use on the degree of roughness and on the capacity. It would be agreed by all interested in hydraulic matters that there was hardly any subject connected with the science of water-supply that was so greatly in need of reliable data as was the question of flow in pipes; and the Coolgardie conduit, with its arrangement of receiving- and regulating-tanks, afforded peculiar facilities for making exact and reliable determinations. There were a number of other points, such as durability and progressive leakage, permanence of pipe-coating, etc., which periodical tests would bring out. So far as Mr. Crowell knew, it was not usual to pay great attention to the procurement of such data; but they would prove of great value if systematically collected and published.

Mr. Fairley Mr. William Fairley thought that the description of the pumping-machinery might have been enlarged with advantage, considering the very important part it played in the success of the works. It would appear that, from the initiation of the scheme, considerable uncertainty was entertained as to exactly how much water would be taken during the first few years after the completion of the works; the quantity having been put as low as 1 million gallons per day. In these circumstances, while it might have been advisable to construct the conduit to the dimensions necessary to deliver the full quantity provided for in the scheme, the necessity for providing the pumping-machinery on a similar scale was not apparent from the information given in the Paper. In the first four stations there were provided three units, each approximately of 300 HP. and equal to one-half of the required capacity, with one of the units, or one-third of the total power, in reserve.

Boiler-plant was provided on a similar scale, and this arrangement appeared to give a safe reserve and to be particularly suited to the case. In the remaining four stations, however, only two units were provided, each equal to the required full capacity, one of the units being spare, with boiler-plant similar. The reason given for this variation was that economy was effected by having the whole of the steam ends of the engines to the same patterns, thereby reducing initial outlay and the number of spare parts to be kept in stock: but in a scheme of such magnitude, where about twenty sets of engines were required, sufficient saving by having one size of engine instead of two or three different sizes, was not apparent. Certainly from the engine-builder's point of view it was convenient, and might mean not only that he realized some little economy in constructing in the shops, but also that power considerably above actual requirements had to be installed in four stations. The advantage of having all spare parts to the same standard for eight stations, spread over 300 miles of country, did not appear to counterbalance the serious expense in providing additional engine- and boiler-power. It did appear that if three units, as in the first four stations, had been provided in the remaining four, giving each station one-third reserve, a very considerable saving would have been effected; and in any case the smaller units for pumping and for steam-generation would have lent themselves more readily to economical working in the early years, should the demand be less than originally estimated—which Mr. Fairley understood was the case in this instance. Instead of 6,068 HP., as shown provided in the Paper, 5,463 HP. would appear to be sufficient if two standard sizes had been adopted instead of one. The difference, at £48 per HP., amounted to about £29,000. The practice of putting down two units, each equal to half the maximum demand, and adding the reserve third as soon as required, would appear to be more economical, and might have been applied in this instance; there would have been considerable saving in the capital cost and also no difficulty in adding further pumping-power as required in the future. On p. 42 the Author mentioned that the feed-water was passed through a heater placed on the exhaust-pipe leading to the condenser. Mr. Fairley would be glad to know if experience had shown any gain from this arrangement. Also, had the Author found any difficulty in keeping each pumping group doing its exact amount of work, due to the tendency in this type of pumping-engine to run short stroke? Mr. Fairley.

Mr. A. Prescott Folwell, of Easton, Pa., remarked that, while it could hardly be questioned that the yield of each catchment-area Mr. Folwell.

Mr. Folwell. was a law unto itself, it was also true that watersheds similarly located had many characteristics in common; and in view of the apparent similarity in several respects between the Helena basin and many of those in the south-western part of the United States, a brief statement concerning these latter might be of interest. Along the Pacific coast of Southern California ran a range of mountains, east of which lay about 80,000 square miles of country, between 4,000 feet and 10,000 feet above sea-level, which had a mean annual rainfall varying between 2·8 inches in the south and 7 inches in the north, with an occasional year of practically no rain at all. As an illustration of the yield in this region, in 1901 one area of 4,000 square miles yielded 0·185 inch, and two others, of about 23,500 square miles each, yielded 0·246 and 0·2785 inch, respectively. In the mountainous region to the west the mean rainfall varied between 5 inches and 60 inches per annum, and several reservoirs were located there. Typical of many of these was the Sweetwater, which was notable in possessing fairly complete records of rainfall, evaporation and yield on the drainage-area, dating back to 1888. The rainfall and the ratio of the yield thereto were given in the following Table, the years extending from September to August inclusive:—

Rainfall and Yield on the Sweetwater Catchment-Basin.

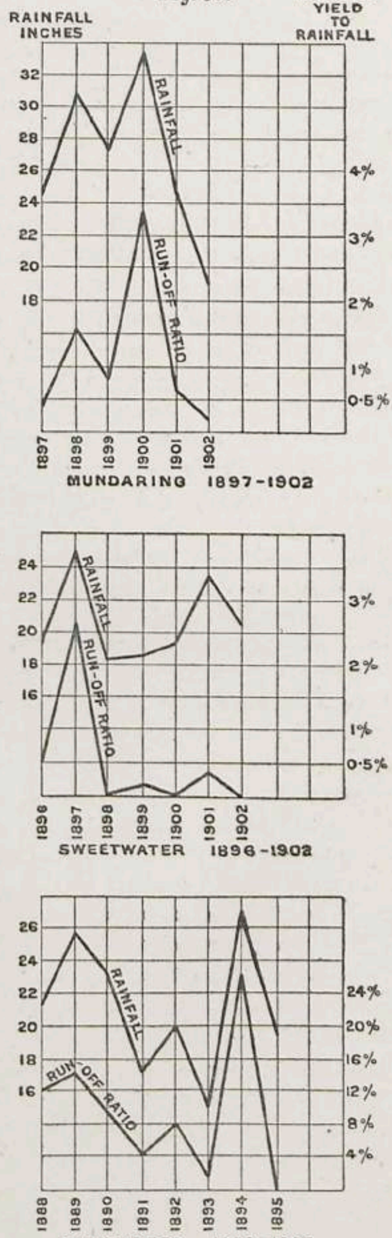
Year.	Rainfall.	Ratio of Yield to Rainfall.	Year.	Rainfall.	Ratio of Yield to Rainfall.
	Inches.	Per Cent.		Inches.	Per Cent.
1888–89	21·00	12·000	1896–97	24·91	2·670
1889–90	25·71	14·000	1897–98	18·18	0·003
1890–91	23.40	9·000	1898–99	18·51	0·135
1891–92	17·14	4·000	1899–1900	19·10	0
1892–93	20·00	8·000	1900–01	23·65	0·368
1893–94	14·76	1·000	1901–02	20·29	0
1894–95	27·14	26·000			
1895–96	19·54	0·517	Mean	20·95	5·55

Practically all the rain fell between September and April, or during about the same length of time as the rainy season at Mundaring; and it was seen to be approximately similar to the rainfall there in both quantity and variations. The percentage running off, however, averaged almost five times as much as at Mundaring. In *Fig. 35* the rainfall and the percentage of yield for each of these two watersheds was plotted, and a remark-

Mr. Folwell.

able similarity between the two was evident, both showing what the Paper referred to, namely, that the ratio of yield to rainfall was even more variable than the rainfall itself; also, that extremes of rainfall were likely to be accompanied by still greater extremes in the yield-ratio, the ratio-curve always moving in the same direction as the rainfall-curve. At Mundaring, however, this ratio had not reached zero, and it was to be hoped that it might never do so; but in all the dry regions of the United States such a condition was likely to occur more or less infrequently. As on the Helena, the high percentages of yield were due to heavy storms, during which one-fourth to two-thirds of the entire season's rainfall fell in 2 or 3 days. The last seven seasons on the Sweetwater conveyed a lesson which would not soon be forgotten in that country, for in only one of these had the yield equalled the evaporation from the reservoir (annually 15 to 20 per cent. of the reservoir's capacity), which had consequently been almost entirely empty for 4 years, although having a capacity of about three times the consumption. It was possible that the apparent similarity between these two cases was but superficial; but if not, the storage allowance at Mundaring, equal to 2 years' consumption, was not

Fig. 35.



Mr. Folwell. only not excessive but might prove disastrously small, and the drainage area itself insufficient, as the Author considered it to be. Calculation based on *Fig. 5* (p. 12) and Table IV. of the Appendix showed that the Mundaring reservoir would apparently lose by evaporation about 780,000,000 gallons per annum when full, and about one-fifth of this amount when half full. If it had been full at the beginning of 1901, and if the daily consumption had been 5,600,000 gallons, it would have been lowered during 1901 by about 1,267,000,000 gallons, and during 1902 by about 185,000,000, leaving less than 1,500,000,000 gallons to tide over a possible third dry year; while two successive years like 1902 would completely empty the reservoir. It would seem probable, therefore, that the fears of the Author were well founded, and that a study of the rainfall and yield of this and adjacent basins should be continued, so that before the consumption reached 5,000,000 gallons per day definite plans for increasing the storage, and especially for increasing the yield, might be decided upon and carried out.

Mr. Fuertes. Mr. JAMES H. FUERTES observed that the yield of streams depended primarily upon rainfall; yet, under ordinary conditions, the fluctuations in yield might be in no definite degree proportional to the fluctuations in rainfall. So far as actual rates of flow were concerned, each stream was a law unto itself. Two streams with similar adjacent watersheds, with the same annual rainfall, and with apparently identical conditions in all respects, might show wide differences in ordinary rates of flow; and attempts to deduce actual monthly flows from rainfall-records could in most cases result only in rough approximations, likely to be in error by anything between 5 per cent. and several hundred per cent., the likelihood of large errors in the percentage being greater with small than with large rainfalls. Within the limits of the United States there existed meteorological and other conditions, which, in different localities, conspired to produce stream-flows ranging from practically the entire rainfall to practically nothing. The Yakima River in the northwest, for instance, had a normal yield considerably in excess of the normal rainfall, as reported, the excess being derived from the melting of the snow in the high mountainous regions. On the other hand, there were streams in the desert lands of the west which were practically dry throughout the greater part of the year. While the conditions indicated by the very low yield of the Helena River and its tributaries were extreme, they had been approached by certain rivers in the United States; and in one at least, the Cimarron River in Colorado and Kansas, the annual rate of off-flow had been very much lower than the Helena gaugings had so far indicated. The Cimarron (with a watershed of 5,200 square miles) discharged

in 1896 only 0·004 inch of rainfall from its watershed, this being only one-tenth the rate of flow shown by the Helena in 1902. Similarly, in 1896 the Arkansas River at Hutchinson, Kan., with a watershed of 34,000 square miles, gave a yield of only 0·06 inch of rain, the average rainfall over the watershed for that year being about 20 inches. These wide variations in rates of flow showed, as the Author stated, the extreme caution necessary in estimating stream-flows in dry countries, where important works would depend for their usefulness and efficiency upon the water collected. The most important point for determination, in countries where the normal yield was small, was the minimum annual flow to be expected for different annual rainfalls below the normal. In such countries the amount of storage necessary was usually so great that actual minimum flows, or minimum monthly flows, were not of much significance. In some cases it was also important to know what the minimum annual flow for two or even three successive years might be. In arid countries it was not unusual to have two consecutive years during which stream-flows might be extremely small. For instance, in 1896, 1897 and 1898 the total annual flows of the Arkansas River at Hutchinson were respectively equal to but 0·06, 0·07 and 0·12 inch of rainfall on the watershed. Viewed in the light of the gaugings quoted hereafter, it was therefore quite probable, as the Author suggested, that the 2 years' storage originally provided in the Mundaring project might not prove sufficient, with the present watershed, to tide over a cycle of dry years and to afford a constant daily supply of 5½ million gallons. The annual precipitation and its seasonal distribution appeared to be the two principal factors determining the annual yield of streams; and, of the two, the seasonal distribution undoubtedly had the most direct effect. For instance, the flow of the Muskingum River in 1888, with 42·6 inches of rain, was 10·3 inches from the watershed, while in 1893, with 42·36 inches of rain, the yield was 16·2 inches, an increase of 57 per cent. with a slightly smaller rainfall. In the first case about 25·5 inches of the total precipitation occurred during the months June–November inclusive, and 17 inches in December–May, while in the second case the conditions were reversed. The only other condition appearing to exert much influence on the yearly flow was the relative capacity of the watershed to absorb water, and subsequently to give it up with a comparatively small evaporative loss. Dense forests and abnormal ground-storage seemed to be favourable to such conditions, for numerous observations supported the view that from areas covered with large dense forests the Mr. Fuertes.

Mr. Fuertes. yield was uniformly larger than from those not so protected. In order to show this effect, however, the forests must be so thick as to afford dense shade, to maintain the atmosphere near the ground in a moist condition, at a comparatively constant temperature, and to prevent the winds from blowing over the ground; ordinary patches of forest-area, second growth, or scrubby timber and bushes, had practically no effect towards increasing stream-flows. The foregoing remarks applied, of course, only to total annual yield, and apparently the size of the watershed, and its geological and topographical features, had little effect on that yield. These views were substantiated by the following data (p. 101) relating to the annual flows of several rivers. The yields tabulated were the smallest recorded flows, resulting from the stated annual rainfalls, which appeared in such records as had come under his notice. For instance, if on any stream a rainfall of 42 inches occurred in several different years, giving different yields, that year had been selected for tabulation which gave the smallest yield, if it was unusually low for the given rainfall; if the flow was not unusually low it had not been tabulated. Thus a given stream might appear in the tabulation only once, or it might appear several times. The yields tabulated were not necessarily, therefore, the minimum flows of the various streams. For instance, the yield of the Muskingum in 1895, with a rainfall of 29·84 inches, was but 4·9 inches; this, however, was a greater relative yield than 10·3 inches in 1888 from a rainfall of 42·6 inches. For this particular stream none of the annual flows, with the exception of that for 1888, were extraordinarily low in proportion to the rainfall. The 1888 gauging alone, therefore, had been given. The tabulation included rivers with watersheds ranging from 18·9 square miles to 54,900 square miles in area, and at elevations varying between the level of the sea and a general altitude of 6,000 or 7,000 feet above sea-level. In mean annual temperature the districts represented varied between 40° F. and about 65° F., and in mean annual rainfall between 10 inches and 55 inches. Topographically, nearly every conceivable type was exemplified: rivers in mountainous districts, in deforested plains, in ordinary rolling country, in arid deserts, and in flat country dotted with lakes. Streams from heavily-forested areas, in the sense previously described, were not included in the tabulation, there being very few streams issuing from such districts in the United States for which long-period gaugings were available. Since in most cases where stream-flows had to be estimated, only rainfall-records were available, all such estimates must be based on comparisons with other streams which had been gauged, and in order to render such comparisons of value, the nature of the streams

Mr. Fuertes.

EXCEPTIONALLY SMALL ANNUAL STREAM-FLOWS RESULTING FROM CERTAIN ANNUAL RAINFALLS, ON VARIOUS WATERSHEDS.

Stream-flows expressed in Inches of Rainfall running off from the Watershed during the Year.

River.	Drainage Area.	Rainfall.	Yield.	Year.
	Square Miles.	Inches.	Inches.	
Arkansas, Hutchinson, Kan. . .	34,000·0	20·4	0·06	1896
,, ,, ,, . .	34,000·0	25·0	0·12	1898
,, Rocky Ford, Col. . . .	11,440·0	16·5	0·64	1898
Broad, Carlton, Ga.	762·0	56·0	21·4	1898
Cimarron, Arkalon, Kan. . . .	5,200·0	15·0	0·004	1896
Cobbosseecontee, Gordiner, Me. .	230·0	47·0	14·0	1892
Lake Cochituate, Mass.	18·9	46·0	14·5	1876
,, ,, ,, ...	18·9	61·5	15·8	1866
,, ,, ,, ...	18·9	58·5	20·0	1869
,, ,, ,, ...	18·9	58·0	22·5	1867
,, ,, ,, ...	18·9	50·5	23·5	1895
,, ,, ,, ...	18·9	56·0	26·5	1898
,, ,, ,, ...	18·9	67·5	27·0	1863
Croton, Croton Dam, N.Y. . . .	339·0	47·8	17·98	1871
,, ,, ,, ,, ...	339·0	56·0	25·1	1882
,, ,, ,, ,, ...	339·0	59·3	28·9	1898
,, ,, ,, ,, ...	339·0	60·3	32·6	1888
Desplaines, Riverside, Ill. . . .	630·0	32·5	3·0	1895
,, ,, ,, ...	630·0	33·5	5·0	1895
Elkhorn, Norfolk, Neb.	2,474·0	26·1	0·94	1897
Genesee, Mt. Morris, N.Y. . . .	1,070·0	40·7	12·80	1896
Green, Blake, Utah	38,200·0	12·5	2·50	1897
Helena, Australia	569·0	19·3	0·04	1902
,, ,, ...	569·0	24·5	0·08	1897
,, ,, ...	569·0	25·0	0·17	1901
,, ,, ...	569·0	27·2	0·23	1899
,, ,, ...	569·0	30·8	0·46	1898
,, ,, ...	569·0	33·3	1·16	1900
,, ,, below weir . .	50·0	35·0	2·30	1899
,, ,, ...	50·0	36·0	3·60	1901
,, ,, ...	50·0	40·0	7·40	1900
James, Buchanan, Va.	2,058·0	41·9	10·8	1896
Jefferson, Sappington, Mont. . .	8,984·0	18·0	0·34	1898
Kansas, Lawrence, Kan. . . .	54,891·0	26·0	0·86	1898
Muskingum Zanesville, O. . . .	5,828·0	42·6	10·3	1888
Mystic, Mass.	26·9	50·4	16·3	1895
Neshaminy, Forks, Pa.	139·3	44·4	16·6	1896
,, ,, ,, ...	139·3	67·0	35·8	1889
Ocmulgee, Macon, Ga.	2,425·0	50·0	15·4	1898
Perkiomen, Frederick, Pa. . . .	152·0	49·7	17·3	1896
,, ,, ,, ...	152·0	66·8	36·1	1889
Potomac, Great Falls, Md. . . .	11,880·0	51·1	15·3	1886
Presumpscott, Sabago Lake . . .	470·0	59·2	23·8	1888
Rio Grande, S. Marcial, N.M. . .	28,067·0	10·0	0·36	1896
Smoky Hill, Ellsworth, Kan. . .	7,980·0	25·0	0·12	1897
,, ,, ,, ,, . .	7,980·0	26·0	0·13	1898
Solomon, Niles, Kan.	6,815·0	25·0	0·29	1898
South Platte, Denver, Col. . . .	3,840·0	11·5	0·61	1896
Sudbury, Mass.	78·0	44·5	16·6	1892
,, ,, ...	78·0	52·1	22·3	1895
Tohickon, Pa.	102·2	68·0	42·0	1889

Mr. Fuertes. used for comparison should be known. It was proper, therefore, in order to render the foregoing data of practical value, to state briefly the general character of the watersheds from which these streams flowed. In these descriptions, the term "deforested" did not necessarily mean entirely devoid of vegetation, but referred to watersheds which might contain a considerable amount of forest-area, if open, or scattered in patches, as already explained.

Arkansas River, above Hutchinson, Kan.—Practically totally deforested. Rises in mountains of Colorado, but principal part of watershed is in the rolling prairies of Colorado and Kansas. Since 1887 large quantities of water have been diverted at the head-waters in Colorado for irrigation in that State. The effect of this diversion, while significant, will not account entirely for the very low flows. Not a very large percentage of watershed under cultivation. Mostly ranges. Elevation generally from 2,000 to 3,000 feet above sea-level.

Cimarron River, above Akalon, Kan.—Totally deforested; at times disappears in ground by percolation at parts of its course, reappearing again above Arkalon. Rate of flow small and generally without much fluctuation. Watershed lies 3,000 to 4,000 feet above sea-level, and is largely devoted to cattle-ranches.

Cobbosseecontee and Presumpscott Rivers, in Maine.—Watersheds comparatively flat land, largely covered with more or less open forests, but large percentage of exposed water-surface in lakes and ponds; about 500 feet above sea-level.

Cochituate Lake, in Massachusetts.—Similar to Sudbury, but a much larger percentage of water-surface and marshy land.

Croton River.—Much broken with high steep hills; considerable second growth and open forest-lands in south-eastern New York, about 500 feet above sea-level. Fair proportion under cultivation.

Desplaines River.—Deforested rolling prairies in Illinois; considerable cultivated areas.

Elkhorn, Kansas, Smoky Hill, and Solomon Rivers.—Streams in Kansas and Nebraska. Rolling prairie country, totally deforested. All 1,000 to 2,000 feet above sea-level. Watershed partly in pasture land, partly under cultivation.

Genesee River, above Mt. Morris, N.Y.—Rolling hilly country in New York and Pennsylvania. Considerable timber on portions of watershed and a good deal under cultivation. Mostly good farming-country. General elevation 1,000 to 2,000 feet above sea-level.

Green River, above Blake, Utah.—Deforested; rises in Rocky Mountains in Wyoming, flowing south to join Colorado. Watershed above Blake, 6,000 to 7,000 feet above sea-level; generally rolling elevated plateaus. Valleys generally very wide and flat, and frequently deeply depressed below the elevated table-lands.

James River, above Buchanan, Va.—In mountains of Virginia. General elevation 1,000 to 2,000 feet; hills and mountains quite generally covered with second-growth timber. Small proportion under cultivation; several broad flat valleys, amounting to perhaps 10 per cent. of watershed, entirely cleared.

Jefferson River, above Sappington, Mont.—At head-waters of Missouri River, in Montana, in mountainous country. Considerable timber and fair proportion of open wide valleys. General elevation 5,000 to 7,000 feet above sea-level.

Muskingum River, above Zanesville, O.—Deforested. Rolling prairies in south-eastern Ohio; a good portion under cultivation, mostly farming-country, about 1,000 feet above sea-level.

Mystic River.—In Massachusetts; generally similar to Sudbury.

Neshaminy, Perkiomen and Tohickon Creeks.—Adjoining watersheds in Pennsylvania; deforested farming-country in rolling hills about 500 feet above sea-level; about 25 miles from Philadelphia. Mr. Fuertes.

Ocmulgee River, above Macon, and *Broad River*, above Carlton.—Streams in Georgia. Considerable timber-land on watersheds, but not sufficiently dense to affect stream-flows noticeably; mostly rolling country 500 to 1,000 feet above sea-level.

Potomac River, above Great Falls, Md.—Generally mountainous and hilly, with broad open valleys; considerable proportion of valleys under cultivation; mountains largely covered with open timber.

Rio Grande River, above S. Marcial.—Deforested; head-waters rising in mountains. More or less timbered land along stream and its tributaries. Large part of watershed in arid plains. General elevation of watershed about 6,000 feet. Some irrigated areas in crops, but most of the country wild.

South Platte River, above Denver, Col.—Deforested; rises in Rocky Mountains. Sometimes goes entirely dry. Lower part of watershed in rolling country. Watershed generally mountainous, 6,000 to 8,000 feet above sea-level.

Sudbury River.—Deforested; rolling farming-country in Massachusetts.

For many of the streams cited records of rainfall and yield had not been kept for a sufficient length of time to give a reliable statement of minimum monthly or daily flow. In the form in which they were given above, however, the data might be of service as a guide in forming a judgment as to probable minimum annual flows where similar conditions might prevail. It would be noted that for most rates of precipitation few of the streams had reached their probable minimum yields. The Croton records, for instance, extended over nearly 37 years, and the Sudbury 27 years, while the Cochituate records went back to 1863. The Cochituate in the sixties, with very heavy rainfalls, showed abnormally low yields, as compared with the yields for 1898; from which it might be implied that, if the Croton and the Sudbury had been gauged at the same time, lower flows would have been recorded for similar rainfalls than had since been reached. It must be borne in mind that for rainfalls of less than about 40 inches most of the gaugings given covered but short-time observations, many of which extended over only very few years; and somewhat smaller relative flows might be expected than those shown. In dealing with such statistics, the attempt to deduce the laws governing the observed phenomena was a fascinating study, but Mr. Fuertes felt that in the present state of knowledge it was better to avoid such temptations, and hence he was content with merely arranging the few observations which had come under his notice in a form convenient for comparison. In reading over the Author's account of the construction of the concrete dam, or weir, Mr. Fuertes had been much impressed with the fact that

Mr. Fuertes. daily temperature-changes of 60° F. to 70° F. were of common occurrence for at least 6 months of the year. This might have had something to do with the absence of subsequent cracks due to changes of temperature. In cold climates monolithic masonry or concrete dams nearly always developed cracks in the upper parts on the advent of cold weather. Concrete expanded and contracted about the same amount, for equal changes of temperature, as steel; but while steel had sufficient strength to resist such strains, if anchored at both ends, concrete of proportions suitable for use in dams had not, being, theoretically, only able to stand a fall in temperature of about 13° F. below that at which it took its final set. This consideration would indicate that in large dams as much of the concrete as practicable should be laid in cool weather.

A new waterworks development for the City of Lynchburg, Va., placed under construction about a year ago, included a pipe-line 21 miles long and a concrete dam across Pedlar River, a tributary of the James River. The present plans were for a supply of 8,000,000 United States gallons[1] per day. The pipe-line traversed very broken country through which hauling with teams was expensive, and, at some seasons, impracticable, except for very light loads; this consideration had had much to do with his recommendations as to the methods and materials to be used in building the pipe-line. As now under construction it was a composite line, 30 inches in internal diameter, consisting of about 18·8 miles of wood-stave pipe, about 1 mile of cast-iron pipe, and 1¼ mile of steel pipe. All straight portions of the steel line were of locking-bar pipe similar in construction and in details of joint to that used for the Coolgardie line, curves being made with riveted steel specials. The line had many summits, as would be understood from the fact that eighty-nine air-valves, four stand-pipes to break the static heads and limit the hydraulic gradient, and two pressure-relief valves, had been provided in the 21 miles. Gate-valves, with by-passes had been placed in the conduit at intervals of about 2 miles. Where these occurred at stand-pipes they were placed on the down-stream side of the stand-pipe, between an air-valve, or cluster of air-valves, and the stand-pipe. Air-valves were placed each side of the gate-valves at all places in the main conduit. Gate-valves located between the stand-pipes permitted accurate regulation of the flow, for any desired delivery, while maintaining the conduit full, under pressure, at all points. There were three

[1] 1 United States gallon = 0·8331 Imperial gallon.

Mr. Fuertes.

tunnels on the line, with an aggregate length of 2,000 feet; in all tunnels the roofs and side walls were to be lined with concrete, and the pipe was to be built continuously through them. Where the line crossed the James River, and at certain other important crossings, as well as in the city streets, heavy cast-iron pipe was to be used; at all other places wood-stave pipe would be employed for heads up to 200 feet, and steel pipe for all heads higher than 200 feet. The wood-stave pipe was built in the trench, working simultaneously at different sections, and joining the different sections of completed pipe by springing in staves cut about $\frac{1}{8}$ inch longer than the spaces to be filled. The staves were of California redwood, in lengths varying between 10 and 22 feet, and averaging 15 feet, not more than 10 per cent. being shorter than 12 feet. They were of perfectly clear stock, and only timber first-class and sound in every respect was accepted. The staves were brought from California ready for use. After arrival at the nearest railway-station they were inspected, then hauled out to the line of work, and distributed along the trench in piles about 3 feet wide and 3 feet high, special efforts being made to have the staves built into the pipe as quickly as possible after delivery. It was impossible to get perfectly dry stock, and when the staves were piled where exposed to the sun and wind, the ends were apt to split and the flat surfaces to splinter more or less. All the stock had been kiln-dried and was what would be called well-seasoned. The pipe was covered over as soon as possible after clamping up the band. The staves were dressed from 2-inch by 6-inch stuff, with outer and inner surfaces true to the proper curves, and were $1\frac{1}{2}$ inch thick after dressing. Both edges were plane and radial to the circles, but one had a very small tongue left longitudinally along its centre, about $\frac{1}{8}$ inch wide and $\frac{1}{16}$ inch high; nineteen or twenty staves completed the circle. Where staves abutted end to end, the joint was made by inserting a thin steel plate, $\frac{1}{8}$ inch thick and $1\frac{1}{2}$ inch wide, in kerfs cut in the ends of the abutting staves. This plate was a little longer than the staves were wide, so that in clamping up the bands the ends of the plate would be jammed into the sides of the staves adjoining. All the plates were coated with preservative coating of the kind used for the bands and shoes. Each band had a formed hemispherical head on one end and threads and nut on the other. The bands were made of homogeneous mild steel, having a tensile strength of 58,000 to 65,000 lbs. per square inch. Each rod was required to be capable of being bent back flat upon itself when cold without

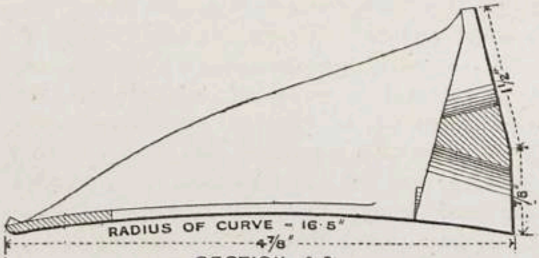
Mr. Fuertes. sign of fracture. The threaded end was not upset, the threads being cold rolled. The specifications required the strength of the headed end and the threaded end to be greater than that of the body of the rod. The rods were shipped from the rolling-mills, laid out straight, and tied in bundles of convenient weight for handling. Rods were taken from each lot at random, and tested to destruction. Before coating, the bands were bent by hand around circular bending-tables, to give them nearly the proper curve to fit the pipe. Each band was then examined carefully, cleaned and freed from rust, and provided with a nut and cut washer $\frac{1}{8}$ inch thick. After bending, the bands were wired together loosely in bundles of about twenty-five, and dipped in a hot bath of melted "Pioneer Mineral Rubber Pipe Coating." This was an asphaltic preparation, a product of "Gilsonite" mined in Utah; and a series of comparative tests conducted by him when seeking a suitable protective covering for the bands and shoes for this pipe-line, had shown it to be practically the only coating, out of several tried, that yielded satisfactory results. He had given the subject much attention, because the life of the bands and shoes would determine the life of the pipe-line. The shoes were coated similarly to the bands. After cooling and drying, the bands and shoes were distributed along the line of the excavations, and piled up for use. The piles were examined immediately after distribution, and all parts of the coating accidentally marred or rubbed off during transportation were repainted. After the bands were on the pipe and the final clamping completed, all the ironwork was then again gone over and retouched, where the coating was imperfect, with a thick coat of Smith's durable metal-coating. The shoes were of malleable cast iron having a tensile strength of 40,000 lbs. per square inch. The quality of the iron was tested occasionally by battering a shoe out flat with the sledge, and observing the action of the metal under this treatment. Altogether, about 320,000 shoes would be used, so that a difference of 1 cent each in the cost represented the sum of $3,200.00 (£657). Considerable care had therefore been taken in the design of the shoe, to produce a form that would have the necessary strength, and at the same time keep the weight down to the lowest limit, without allowing the metal to be less than $\frac{1}{8}$ inch in any part. The details of the shoe as finally designed were shown in *Figs. 36*. The shoes weighed, after coating, almost exactly 1 lb. each. They had proved very satisfactory from a practical point of view, as there was no tendency for the bands to jump out

of the shoes when turning up the nuts, nor for the points of the Mr. Fuertes. shoes to dig down into the staves when the shoes slipped around the pipe with the tightening of the bands. Whenever the head on pipe was more than 50 feet the bands were spaced according to the following formula:—

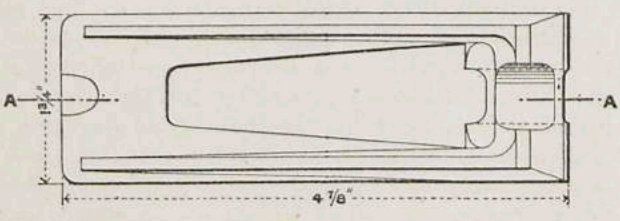
$$N = \frac{330\ HD}{d^2\ S}$$

in which N = number of bands per hundred feet of pipe; H = head on pipe in feet; D = diameter of pipe in inches; S = permissible

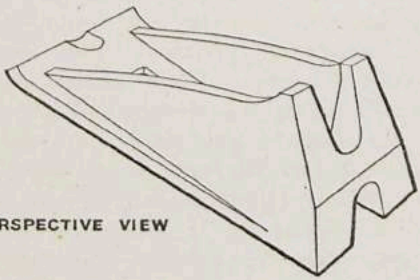
Figs. 36.



SECTION AA



PLAN



PERSPECTIVE VIEW

stress on bands in pounds per square inch of section; d = diameter of bands in inches. In this case, with S = 12,000 lbs. per square

Mr. Fuertes inch, the formula reduced to $N = 3 \cdot 3\ H$. Where the head was less than 50 feet, the bands were spaced closer together than the formula would require, and in accordance with the following Table:—

Head, 0 – 20 ft.	108 bands per 100 feet.
„ 21 – 26 „	114 „ „ „ „
„ 27 – 31 „	120 „ „ „ „
„ 32 – 36 „	127 „ „ „ „
„ 37 – 40 „	134 „ „ „ „
„ 41 – 45 „	150 „ „ „ „
„ 46 – 50 „	165 „ „ „ „

The entire pipe was laid under ground with a cover of earth about 2 feet 6 inches deep. In placing the bands they were first strung over the end of the pipe and accurately spaced, brace-wrenches with a 6-inch sweep being used to bring them quickly to a light tension. While tightening the bands, the staves were coopered with wooden mallets from the inside, and the bands were tapped lightly to allow them to slip round the staves as tightening proceeded. The spacing-gang was followed by another, which completed the clamping, using S wrenches with 10-inch handles as a precaution against over-tightening. The shoes were staggered according to a uniform system, and lay around the top half of the circumference of the pipe. Where wood-stave joined cast-iron pipe, special bell-ends 7 inches deep were cast on the iron pipe and the stave-pipe was inserted therein, the joint being packed with oakum, run with melted lead and caulked in the manner customary in jointing cast-iron pipes. The 30-inch valves also were cast with large bells, and the joints were made in the same way. At the crossing of the James River the pipe was under a head of 410 feet, necessitating liberal provision of air-valves in case the pipe should be ruptured in the valley. Clusters of valves, with sufficient capacity to prevent the reduction of pressure within the pipe from exceeding about 8 lbs. per square inch, under the maximum conditions, were provided at the summits on each side of such places. Vertical and horizontal curves were used, the minimum radius adopted being 200 feet. No difficulty was experienced in springing the pipe to this radius, the staves on the outside of the curve being sledged back endways to make tight butt joints before final clamping. The great advantage accruing from the use of wood-stave pipe of large size, in the matter of cost, was exhibited by the following average prices per lineal foot bid by the successful contractor for wood-stave, cast-iron and steel pipes, 30 inches in diameter each, erected, but not including excavation, which was tendered for separately. The prices for wood-stave pipe

were, for the staves, in place, and for the bands, per hundred bands in place. The average head on the wood-stave portion was 110 feet, requiring 363 bands per hundred feet. Mr. Fuertes.

	Per Lineal Foot.
Wood-stave pipe	\$2·29 (9*s.* 5*d.*)
Steel pipe $\frac{1}{4}$ inch thick	\$4·00 (16*s.* 5*d.*)
" " $\frac{5}{16}$ " "	\$5·00 (20*s.* 6*d.*)
Cast-iron pipe	\$6·38 (26*s.* 2*d.*)

For higher heads than 200 feet it would be necessary to use thicker staves and heavier bands. For a wood-stave pipe to be successful, the hydraulic gradient must be established with great care, so that the pipe would always be under pressure at all points and for all rates of flow. If this precaution were neglected, the staves in those positions where the pipe was only partially filled would decay rapidly; but when such a pipe was properly laid out, and properly built, its life should be equal to that of other forms of conduit in general use. The advantages in its use lay in the ease with which the materials could be transported through rough, broken country, the rapidity and simplicity of construction, cheapness, and the fact that the cost varied with the head under which the pipe was to work, thus making it possible to cheapen the cost under small heads by spacing the bands farther apart. Pipes of this type had been used largely in the western parts of the United States for many years, mainly in the development of irrigation-works; and, when properly built, they had proved eminently satisfactory and durable. The oldest stave-pipe in use, so far as he was aware, was at Wicopee, on Fishkill Creek, in New York State, where a mill-flume, 30 inches in diameter, 300 feet long, banded with flat hoops, and built in 1850, was still in service and in good condition. In connection with the City waterworks of Denver, Col., there were more than 100 miles of such pipes, the first having been laid in 1889. Stave-pipes, banded with round hoops and using shoes of the style adopted for the Lynchburg pipe-line, were the invention of Mr. C. P. Allen, when City Engineer of Denver, Colorado; and the United States patents having expired, the invention was now public property. The use of wood-stave pipe instead of cast iron for the Lynchburg pipe-line had involved a saving of about \$350,000, nearly \$17,000 (£3,485) per mile on the entire length, or \$19,000 (£3,895) per mile on the length for which the substitution was made. This saving in cost, as well as in interest and sinking-fund charges, would enable the town, in about 20 years, to build a second conduit of equal capacity, thus doubling the supply, at a total cost for both lines

Mr. Fuertes. of about two-thirds of the cost of a single line of cast-iron pipe with a capacity equal to that of one of the wood-stave lines. It was also claimed, as a result of measurements made several years ago, that wood-stave pipes would deliver larger quantities of water than cast-iron pipes, under similar conditions as to sizes and gradients; but he thought this point should not be given too much weight. He considered the greatest advantage of the wood-stave pipe in respect to delivery to lie in its ability to continue its original rate of discharge after long usage, due to the fact that corrosion and incrustation did not take place, and hence a smooth interior surface was maintained.

Mr. Goument. Mr. C. E. GOUMENT considered that, judging from the figures of rainfall and discharges for the years 1897–1902, it seemed doubtful whether the storage-capacity, enormous as it was, would be sufficient to tide over three consecutive dry years, such as 1897, 1901, and 1902 appeared to have been. In India drought generally occurred in periods of 3 years, and if the same conditions held good for Australia, the reservoir would certainly need enlargement if shortage was to be avoided. The rainfall on the areas immediately above and below the weir, and the yield from them, were more favourable than those of the large catchment-area which had actually been utilized; and it seemed to him that a substantial saving might have been effected in the initial outlay on the storage-reservoir, and a more certain supply secured, if the recommendation now made by the Author, to extend catchwater drains into the well-watered and quick-shedding country draining into the Helena below the present weir, had been carried out in the first instance. A smaller reservoir would probably have sufficed. With regard to the class of pipes used for the supply-line, considering the enormous expenditure incurred on this item of work and the fact that the life of thin steel pipes was comparatively short, it seemed open to question whether, financially, it had been a sound decision to use these pipes in preference to ordinary cast-iron pipes which had a much longer life under the same conditions. His experience led him to think that there was no comparison between the two as regarded durability. The life of a thin steel pipe depended to a large extent on the time the coating lasted. Once this was worn through, the pipe soon corroded and became useless. In 1892 some steel tubes, $\frac{1}{4}$ inch thick, were laid down on the rising-main of one of the pumping-stations in the Punjab where the pressures were too heavy for ordinary cast-iron pipes. Some of these pipes had already corroded internally and had recently had to be renewed. Chemical analysis showed no acid in the water which would account for

excessive corrosion. The tubes had been obtained from a firm of good repute, and the coating had seemed to be in sound condition when they were laid 13 years ago. On the other hand, some cast-iron pipes laid 25 years ago in the same town had been taken up recently and had been found to be in an excellent state of preservation. Some figures to show to what extent cast iron would have been more expensive than steel would have been useful. In India, as a rule, imported cast-iron pipes, for the pressures for which they were suitable, up to a working-head of, say, about 200 feet, cost but little more than steel pipes, even in places in the Punjab where the transport by land from the nearest seaport was over 1,000 miles of railway. Mr. Goument.

Mr. ALLEN HAZEN, of New York, thought the Paper described conditions entirely unique in waterworks practice. Probably nowhere in the world had water been carried so far, and at such heavy expense in proportion to the quantity. On the other hand, the necessity seemed to have been great, and to have fully justified the project; and the undertaking was an unusually useful one. There were many novel points in the construction, among the most interesting being the use of an entirely new form of pipe. Apparently the weight of the locking bars used for holding the steel sheets together was fully equal to the weight of the steel required for the lap joints and rivets in ordinary riveted steel pipe, so that there was no saving in the total weight. The greater strength of the joint, however, and particularly the greater smoothness of the inside surface, which resulted in less friction of the water flowing in the pipe, were substantial points in its favour, and would seem to ensure to this type of pipe a wide field of usefulness. The lead joints provided at short intervals must act as expansion-joints. American riveted steel pipe-lines were riveted continuously, and American readers would question whether lead joints were really necessary. Changes of 45° or 50° F. in the temperature of water and buried pipes were common in the northern United States; and computations indicated that there would be no serious trouble from the lack of expansion-joints, even though the range of variation of temperature were considerably wider than this. The use of the sleeves and lead joints resulted in greater smoothness of the interior surface, however, and the arrangement seemed to have been convenient and satisfactory in this case. It might not be so readily applicable to the much larger pipes which had been extensively used in waterworks in recent years. The impounding-reservoir was in a region with much greater rainfall than the region served with water. Nevertheless, the record of yield indicated that, Mr. Hazen.

Mr. Hazen. generally speaking, even this region was at times exceedingly dry; and it appeared that to ensure a supply, water must be carried in storage for unusually long periods, that was, for 2 years and more. Under these circumstances, particularly in a warm and sunny climate, it would seem, from American experience, that the water in this reservoir would be sure to become foul from the growth of algæ and other small plants and animals. It would be interesting to hear what the experience had been in this respect. The passage of the water through the pumps, pipes and reservoirs would no doubt tend strongly to remove such tastes and odours if they existed at the source. The water apparently entered each successive reservoir through a pipe extending above the water-level, and so fell into the reservoir through the air. The water would thus be aerated successively at each step in its progress. With water passing through the pipe and reservoirs at the full nominal rate, about 12 days would be required for it to pass from its source to the point of use. Under the present conditions, with only one-half to one-quarter of the full quantity of water passing, a month or more would be required, and the opportunities for purification were such as did not often occur in waterworks practice. The tastes and odours which might be most troublesome, and the removal of which would be most likely to be effected by this process, could only be determined by direct observation, and their presence or absence was not indicated by the Tables of analyses in the Paper. The cost of the work was surprisingly low. Although a large part of the steel came from the United States, the cost would have been low had the work been done there, instead of on the other side of the globe. The completion of a large enterprise at so reasonable a figure, and in a way so satisfactory in other respects, reflected great credit upon both the engineering and the business ability of those who had had the matter in charge.

Mr. Herbert. Mr. Frank Herbert, of Glasgow, thought that while the Author modified by subsequent comparisons his statement that the price of locking-bar pipes was a little more than that of riveted pipes, he did not sufficiently emphasize the fact that on the basis of such comparisons the locking-bar pipe was actually cheaper. It had an ultimate tensile strength exceeding 27 tons per square inch of plate section, whereas the best double-riveted joint only had an ultimate tensile strength of 19 tons per square inch; and allowing the same factor of safety for both, and adding $\frac{1}{16}$ inch for corrosion, the weight of the one was not more than three-quarters of that of the other; moreover, owing to the lap joints and rivets, the riveted pipe required to be $7\frac{1}{2}$ per cent. larger in diameter to

give the same frictional loss, and as Mephan Ferguson pipe could be made rather cheaper per ton than riveted pipe, for equal efficiency as a pressure-main, Ferguson pipe cost about two-thirds the price of riveted pipe. It would be noticed, from the actual tests described on p. 27, that the loss of water through 295 miles of main, including nine pumping-stations and nine large reservoirs, amounted to less than 3 per cent., allowing the full delivery of 5 million gallons per diem: which spoke volumes for the efficiency of the pipes and joints. No figures were given to show the basis on which the diameter of the Coolgardie pipe-line had been determined, and as this was an important matter, it might be interesting to analyse it. From figures given by the Author (which could, of course, have been ascertained approximately when determining the size of pipes required), the cost of the pumping-plant, including spares, erected, was equal to £61·25 per horse-power. The charge for fuel at 32*s.* per ton (Newcastle, N.S.W.), assumed to give 74 millions duty in every-day work = £19·71 per foot of head pumped against per annum at 5 million gallons supply per day; and each foot of head cost £64·44 for pumping-plant. Each 1 inch diameter of pipe cost erected £62,333. The friction through 307½ miles of pipes delivering 5 million gallons per day (3,472 gallons per minute) would be— Mr. Herbert.

			Difference
For 32-inch pipes,	1 in 2,420	671 feet	
			111
31 " "	1 in 2,075	782 "	
			146
30 " "	1 in 1,750	928 "	
			169
29 " "	1 in 1,480	1,097 "	
			215
28 " "	1 in 1,230	1,312 "	

The saving in cost of pipes by reducing from 32 inches to 31 inches would be £62,333; while the loss by extra cost of pumping-plant would be £64·44 × 111 = £7,153 + extra annual cost of fuel = £19·71 × 111 = £2,088, which latter capitalized at 5 per cent., 7½ per cent., or 10 per cent. = £41,760, £31,320 or, £20,880 respectively. Thus the saving by reducing from 32 inches to 31 inches would be £62,333, while the loss would be, at 5 per cent. £48,913, at 7½ per cent. £38,473, and at 10 per cent. £28,033. Similarly, the saving by reduction from 31 inches to 30 inches would be £62,333, and the loss would be £66,968, £52,578, or £38,188. The saving by reduction from 30 inches to 29 inches would be £62,333, and the loss would be £77,510, £60,850, or £44,200. The saving by reduction from 29 inches to 28 inches would be £62,333, and the

Mr. Herbert. loss would be £98,615, £77,425, or £56,235. Thus for a supply of 5 million gallons, and allowing 5 per cent. interest, the most economical size was 31 inches, at $7\frac{1}{2}$ per cent. 29 inches, and at 10 per cent. 28 inches. The rate of interest to be allowed depended on the probable life of the goldfields, and the probable lapse of time before the 5-million-gallon supply was required. From the Author's remarks on p. 6 it appeared that the actual demand up to the present had been somewhat disappointing, and that it would have been well to have allowed, say, $7\frac{1}{2}$ per cent., and thus save £62,333 in the cost of the pipe; but otherwise the size adopted appeared to be about the best, allowing as it did about 6 per cent. interest. For a permanent town water-supply in England a 4 per cent. basis might safely be assumed, while for an individual mine or speculative venture 10 per cent. would be better policy. The fact of this, the longest water-main in the world, being of steel, invoked discussion on what had become of late years a much-debated point, namely, the relative life of cast-iron and of wrought-steel or iron pipes. Mr. Herbert's own opinion was that very little corrosion occurred in either cast or wrought pipes when placed in natural ground, generally clay; but that in artificial ground, particularly ashes, slag, or any decomposing material, rapid corrosion took place in both, and generally much more rapidly in cast-iron pipes. In bad ground the life of the pipe was almost entirely dependent on the efficiency of the coating. Tar and asphalt appeared to take a much better hold on wrought pipes, probably because these thin pipes quickly attained the temperature of the bath, even if they were not previously heated, which was sometimes done. He believed that within the next 10 years wrought-iron pipes would supersede cast pipes almost entirely for water and gas-mains, and in many places would be used for sewage.

Mr. Hering. Mr. Rudolph Hering observed that the Paper was of special interest to American engineers, owing to some similarity between the meteorological conditions of the territory near Coolgardie and those of the Pacific coast of the United States. The rainfall in California varied between $3\frac{1}{2}$ inches and 42 inches in one year. The prices paid for water near Coolgardie, even at the lowest figure, 25*s*. per thousand gallons, were much higher than the prices paid on the Pacific coast, due partly to the apparently greater difficulties of construction, and partly to the more elaborate structures than were customary in the pioneer works of California. The Coolgardie scheme appeared to be a thorough and efficient solution of the problem, and the work contained a number of suggestions as to

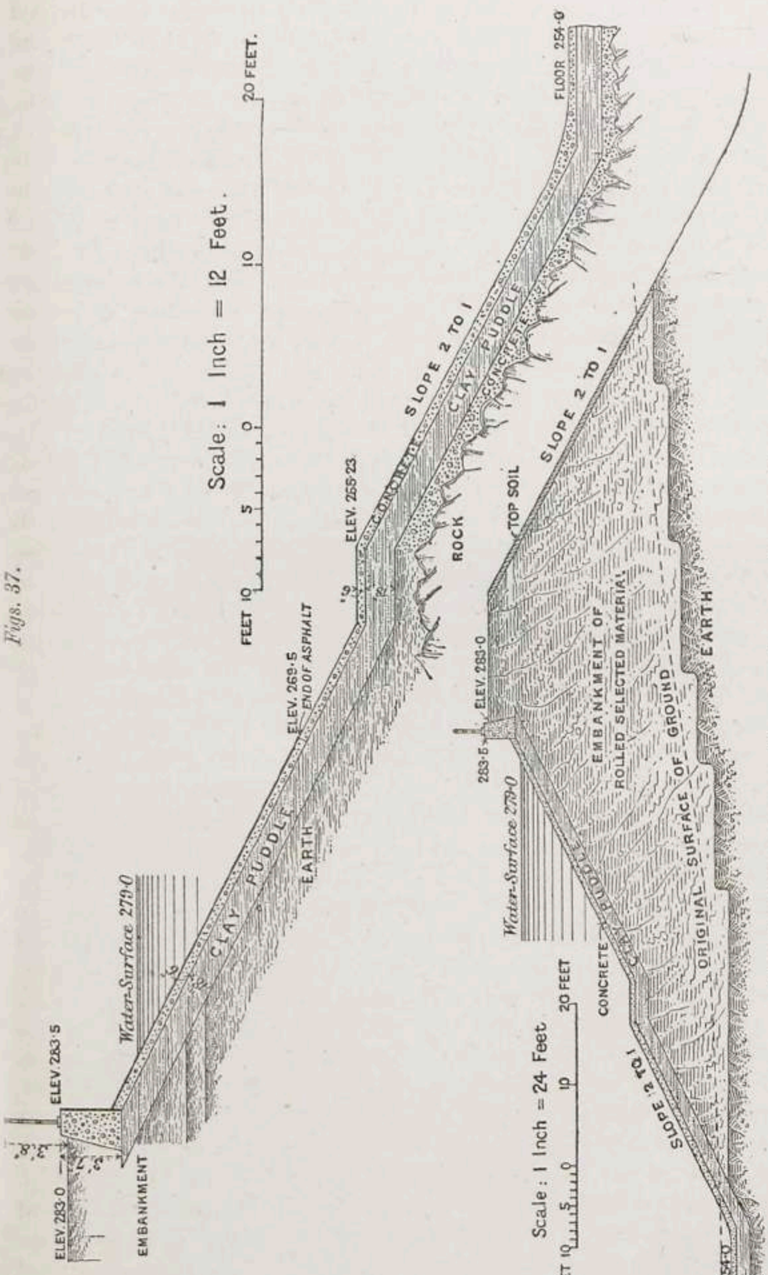
details which might profitably be adopted elsewhere. It was not often that 5,000,000 gallons of water must be elevated daily to a point over 1,000 feet higher than, and over 350 miles from, the source, requiring a total dynamic lift at the pumping-station of about 1,700 feet. In this respect the work appeared to be unique. Although practically all the materials for construction had been imported, the work had required less than 5 years to build, which under the circumstances did not seem excessive for its magnitude and the difficulties of the situation. The rainfall-records, as had been the case in California when that State was first developed, were meagre, and much depended on good reasoning. In arid regions the ratio of yield to rainfall varied greatly, being large for large precipitations and small for small ones. In California, with rainfalls less than 10 inches per annum, unless some of the rain fell as sudden showers, there was usually no off-flow at all. San Francisco was obliged for that reason to store its water for 2 and even 3 years. A few years ago the evaporation during 1 year from the storage-reservoirs was greater than the rainfall or the inflow from the catchment-area. In the western deserts of the United States, that portion of the rainfall which did not evaporate percolated to a considerable depth, and continuously fed the rivers, which, like the Colorado River, discharged large quantities of water through the year, although flowing in the midst of a desert. Naturally, such conditions had invited irrigation schemes, which, near the mouth of such a river, would allow the entire flow to be diverted without infringing upon riparian rights. With regard to the quality of the Coolgardie water, the difficulties due to common salt would gradually disappear, particularly if flat and swampy areas on the catchment-area were sub-drained so as to prevent the capillary action near the surface in dry weather from raising the dissolved salts and holding them upon the surface by subsequent evaporation until they were again dissolved by the rain-water. He could not understand how there could be anaerobic bacterial action in the pipes carrying potable water, which should contain a large amount of oxygen. The descriptions of the locking-bar pipe and of the caulking machinery, which seemed to present novelties of value, were of much interest. Mr. Hering.

Mr. Henry C. Hill, of Philadelphia, communicated an account of the reservoir at the Belmont filtering-station, forming part of the works for the improvement, extension and filtration of the water-supply for the City of Philadelphia. This station was situated about $1\frac{1}{2}$ mile west of the Schuylkill River, and consisted of a settling-reservoir, eighteen covered $\frac{3}{4}$-acre filters, and Mr. H. C. Hill.

Mr. H. C. Hill. a covered clear-water basin. The settling-reservoir was divided by an embankment into two compartments, having an aggregate capacity, with 25 feet depth of water, of 72 million gallons. The reservoir was situated on a hillside, mainly in excavation. The area of the east division at the flow-line was 5·4 acres, that of the west division being 5·33 acres. The east division was constructed about equally of cut and fill, while the west division was nearly all in excavation, principally a quartz-like trap rock with some micaceous rock. In making embankments the most impervious materials of excavation were placed next the inner slope, and all materials were rolled in layers not exceeding 6 inches in thickness. Whenever excavation in rock revealed fissures, these were filled with Portland-cement grout, and the irregular surfaces of the rock in the floors and slopes were levelled up partly with clay puddle, and partly with Portland-cement concrete. In preparing the foundation for the rolled embankment all top soil was thoroughly stripped off, and the inclined ground was stepped in horizontal terraces. All embankments and fills were rolled with four 25-HP. traction-engines, each weighing 10 tons or 3,300 lbs. per foot width of roller-wheels; two 18-HP. traction-engines of 8 tons each; and one 10-ton traction-roller. All the rollers were provided with grooved front wheels and cleated rear wheels. Each division of the reservoir was lined on the floor and slope with 18 inches of clay puddle, on which was placed a lining of Portland-cement concrete 6 inches thick (*Figs. 37*). On the concrete lining over the whole floor, and extending up the slopes to a point 10 feet vertically below the water-line, was placed a $\frac{3}{4}$-inch mixture of asphalt mastic and grit. In order to prevent slipping of the asphalt mixture on the slopes, the concrete was indented with grooves $\frac{1}{2}$ inch wide and deep, and 4 inches apart. At the end of the dividing embankment was a valve-house, in which were placed all the valves for controlling the water to and from the two divisions, which were provided with separate inlet- and outlet-pipes, and screen-chambers. As the reservoirs were designed as settling-basins and not as storage-basins, the water was pumped through a 48-inch cast-iron main, laid on the floors and extending diagonally across each division of the reservoir, and was discharged through a series of 48-inch spigot-and-faucet tees, set at alternate angles of 45 degrees from the vertical. The ends of the inflow-pipes were provided with 48-inch upturned elbows. The water passed upwards from the tees and elbows diagonally across the basin to a floating discharge-pipe, consisting of a 48-inch riveted steel pipe $\frac{1}{8}$ inch thick, provided

Mr. H. C. Hill.

Figs. 37.



SETTLING-RESERVOIR AT BELMONT FILTERING-STATION, PHILADELPHIA.

Mr. H. C. Hill. with a swinging joint at the bottom, and sustained in an inclined position by a cylindrical iron float which kept the mouth of the pipe about 2 feet below the surface, so as to admit of drawing off the upper stratum of water. When both divisions of the basin were in service the water was forced to pass diagonally across and upwards through both divisions, before it left the sedimentation-reservoirs on its way to the filters. The reservoir was designed to be water-tight within the limits of the materials employed, and information had been sought on this point, with the result that very few data had been found to be available on the subject of the water-tightness of puddle and concrete-lined structures. Such tests as might have been made elsewhere were not readily accessible in publications, and the leakage assumed to be properly allowable was therefore entirely a matter of judgment. Obviously, structures like earthen embankments lined with concrete, made up of many separate floor- and slope-sections, and containing many thousand lineal feet of comparatively shallow joints, could not, even with the utmost care in construction, be expected to be absolutely water-tight. The puddle used in the works at Philadelphia was composed of equal parts by volume of rich iron clay and broken stone or gravel, moistened with water and thoroughly mixed in a Chambers pug-mill. As much care was exercised in the testing of clays, and in supervising the proportions and mixing of clay puddle, as in the proportioning of materials and the mixing of concrete. All puddle clays were tested by the Ulzer method of rational analyses, and it was found after a long series of experiments that the best results were obtained with clays containing at least 50 per cent. of available clay constituents, or, in other words, the resulting mixture of 50 per cent. of clay and 50 per cent. of broken stone, would contain 25 per cent. of available clay constituent, as shown by rational analyses. The concrete lining was composed of 105½ lbs. of Portland cement, 3 cubic feet of sand, and 5 cubic feet of broken stone, the stone ranging in size from ¼ inch to 1¼ inch. In some cases, to ensure more water-tight work, the 3 cubic feet of sand was altered to 1½ cubic foot of sand and 1½ cubic foot of limestone screenings. All concrete was mixed in cubical box mixers of at least 1 cubic yard capacity. The dry ingredients, after being dumped in the mixer, were given eight or ten turns, water was then added, and the mixer was given about twenty-five more turns. All cement was required to fulfil the following requirements before it was incorporated in the work :—

Mr. H. C. Hill.

Specific gravity	3·10
Fineness: residue on—	
50-mesh sieve	0·0 per cent.
100-mesh "	10·0 "
200-mesh "	25·0 "
Anhydrous sulphuric acid	1·75 "
Initial set (Vicat needle)	20 minutes
Briquettes 1 square inch in cross section to develop the following tensile strengths—	
7 days (1 day in air, 6 days in water), 1 part of cement, 3 parts of standard quartz sand	170 lbs. per sq. in.
28 days (1 day in air, 27 days in water), 1 part of cement, 3 parts of standard quartz sand	240 lbs. per sq. in.

All cement to be subjected to the boiling test before acceptance.

Six-inch cubes of concrete taken from batches used in the work were required to withstand the following crushing-loads:—

At 30 days old	1,700 lbs. per square inch.
" 60 " "	2,100 " " " "
" 90 " "	2,400 " " " "

It was found in practice that the above requirements were readily met, in fact there were quite a number of 60-day cubes which took a crushing-load of 5,000 lbs. or more per square inch, the average of all cubes being about 3,300 lbs. While the clay puddle was relied on for water-tightness, it had been thought advisable to place on the surface of the concrete lining a ¾-inch layer of asphaltic mixture. Owing to expansion and contraction during rapid changes of temperature, it was doubtful if the layer of asphalt had contributed much to the water-tightness of the structures. Concrete filter-tanks and clear-water basins, with a lining of puddle under the floor and around the walls, had been proved on a 14-day test, with the rise or fall of the water-level measured with hook-gauges, to be practically water-tight. In these structures, of course, the asphaltic lining was not used, nor had experience shown that it was necessary to secure water-tight work. The composition of the asphaltic lining called for in the specification was—

	Parts by Weight.
Neuchatel mastic	70
Bermudez asphalt	10
Clean, sharp grit and sand	20

This mixture would contain about 21 per cent. of total bitumen. The materials were heated in a kettle to about 280° F., and thoroughly mixed. It was found upon test that the mixture specified contained too high a percentage of bitumen, and was too

Mr. H. C. Hill. soft for use on the slopes when exposed to the sun. Experiments were then made to ascertain what percentage of total bitumen in the mass would hold on the slopes without creeping when warmed by the rays of the sun. Concrete slabs about 2 feet wide and 4 feet long, placed at the same slope as the sides of the reservoirs, were covered with mixtures containing various percentages of bitumen, and maintained at a temperature of 100° F. Wires were placed across and above the slabs, and tacks were driven in the surface showing the downward movement of the asphalt. Owing to the fact that it was practically impossible to procure Neuchatel mastic in the Philadelphia market, except in very small quantities, Seyssel mastic was substituted, the above tests indicating that it was in all respects equal to Neuchatel. The several mixtures used contained as an average the following weights of materials for each batch :—

Material.	Floor and First Layer of Slope.	Second Layer of Slope.
	Lbs.	Lbs.
Seyssel mastic	585	598
Grit	315	332
Refined Trinidad asphalt	50	33
Refined Bermudez asphalt	50	37

These mixtures gave by analyses an average of 15·5 per cent. of total bitumen for the first mixture, and 13·2 per cent. of bitumen for the second mixture; the slopes requiring, of course, a stiffer mixture to prevent creeping during hot weather. The water-tightness of all structures forming part of the works for the improvement, extension and filtration of the water-supply of Philadelphia, was a requirement of the specification, and searching tests of each structure were made to show compliance therewith. Upon trial it was found that the west division of the reservoir did not come within the limits fixed by the Bureau of Filtration; the water was drawn off, the surface of the concrete above the line of asphalt was thoroughly dried, and several coats of silicate of soda (syrup) were then applied with an ordinary whitewash-brush. Each coat of silicate of soda was allowed to dry thoroughly before the next coat was applied, and this treatment was found to be very effective in reducing leakage through the concrete. The leakage of the west division of the Belmont reservoir upon final test was 36,000 gallons per day of 24 hours, or about 0·1 per cent. of its cubic contents.

This leakage, after allowing for evaporation, represented a fall of 0·02 foot per day in the water-level. The leakage of the east division of the reservoir upon final test was 21,000 gallons per day of 24 hours, or about 0·06 per cent. of its cubic contents. This leakage, after allowing for evaporation, represented a fall of 0·011 foot per day in the water-level. Mr. H. C. Hill.

Mr. JOHN W. HILL, of Philadelphia, felt that the Author, as well as his eminent predecessor, Mr. O'Connor, and his assistants, were entitled to great credit for the conception and successful completion of so bold an enterprise as the Coolgardie water-supply, which certainly deserved more than passing notice. Personally, he was chiefly interested in the pipe-line, presenting as it did novel features in the construction of steel-plate pipe, and suggesting the probable extended use of a new kind of steel pipe, in which might be developed the full strength of the steel plate within the elastic limit. A little more than 2 years ago he had occasion to prepare the plans and specification for Contract No. 28, known as "Lardner's Point pipe distributing-system," one of more than seventy contracts forming part of the works for the improvement, extension and filtration of the water-supply of the City of Philadelphia. Some thought was given to the possible use of locking-bar pipe such as was used in the Coolgardie scheme; but there being then no manufacturer in the United States who had had any experience in the production of pipe of large diameter of this pattern, and the information now contained in the Paper with reference to the Coolgardie pipe not being available, it was not given the same consideration as it would have if the matter were to come before him at the present time. The pipe-system embraced in Contract No. 28 consisted of four lines of 60-inch pipe, reaching from Lardner's Point pumping-station, the principal pumping-station of the new Philadelphia waterworks, to valve-chambers Nos. 8, 9, and 10 at the western terminus of the pipe-line at Torresdale and Kensington Avenues, a distance of approximately 14,606 feet. While it was contemplated to lay four lines of such pipe for the whole distance, for the present four lines were laid only along Robbins Street, a distance of 800 feet from the pumping-station; and from this point three parallel lines of pipe were laid along Tacony Street and Torresdale Avenue to valve-chambers Nos. 8, 9 and 10, a distance of about 13,802 feet. In taking tenders for this pipe due consideration was given to steel pipe, with and without double-riveted butt joints, and to cast-iron pipe. The conditions favouring steel pipe were:— Mr. J. W. Hill.

Mr. J. W. Hill.

1. Ductility of material, and capacity to resist shocks.
2. Rapidity with which the sections of the pipe could be furnished and made up in place in the trench.

The conditions favouring cast-iron pipe were:—

1. Known durability in service (cast-iron pipe had been used in Philadelphia for a period of over 85 years, and except tuberculation of the interior from the iron and lime salts in the Schuylkill river-water, had apparently suffered no serious injury in strength or carrying capacity during this time).
2. Greater smoothness of the interior of the pipe, and consequent reduction of frictional resistance, and increase of carrying-capacity.
3. Convenience of repair in case of rupture in service.
4. In laying such pipe provision could readily be made for expansion and contraction due to temperature-changes, although these, at the depth at which the pipe was laid after filling the trench, were necessarily slight.

Still another advantage sometimes found with cast-iron pipe, laid in the 12-foot length usual in the United States, was that considerable settlement of the bottom of the trench might occur without rupturing the pipe, or creating serious leaks at the lead joints. The terms of the contract under which tenders were received for both riveted steel and cast-iron pipe provided that the sections of the former should be subjected to a hydrostatic test of 180 lbs. per square inch at the place of manufacture, for such length of time as the engineer might deem desirable, to prove the tightness of the riveted and caulked joints. After the sections were made up in the pipe-trench, and before refilling it, they were to be again subjected to a hydrostatic pressure of 160 lbs. per square inch under the same conditions as to length of time of test. The contract further provided that each joint should be absolutely water-tight under these tests. The provisions with reference to cast-iron pipe were somewhat different. After a pipe had been inspected for casting and foundry-work, and carefully checked as to dimensions, thickness of material in the barrel, eccentricity of bore, and diameters of ends, it was to be dipped in the customary bath of hot pitch and linseed-oil, and when the coating had become dry and cold, the pipe was to be subjected in the proving-press at the foundry to an internal hydrostatic pressure of 300 lbs. per square inch for not less than 10 minutes. While

under this pressure it was to be subjected to a thorough hammer-test, and any pipe which showed leaking or sweating at the end of this time was to be rejected. After the pipe was made up in the usual lengths of 12 feet in the trench, the contract required that it should be subjected to a further pressure of 200 lbs. per square inch for a period of not less than 5 hours, this length of time being fixed to enable the inspectors to examine carefully each joint and length of pipe, and if there should be spongy places in the pipe-barrel which had escaped inspection at the place of manufacture, this time was deemed sufficient to bring the water to the surface and enable such spots to be marked and the proper remedy applied. The test in the pipe-line also permitted of a trial of such forms of special castings as could not be conveniently tested at the foundry. All special castings in this contract, excepting curves and man-hole castings, were 48 inches to 60 inches in diameter, and were made up in the valve-chambers with flanged joints. Tees and crosses were flanged castings. Forty-eight-inch to 60-inch taper-pipes were constructed with a flange on the small end and a socket on the large end to join with the spigot of a 60-inch pipe; and all straight castings were tested at the foundry under the same conditions as applied to the cast-iron pipe. At the time this contract was made there was only one hydrostatic pipe-press in the United States of sufficient capacity to apply the required test, and the two other foundries which were employed in the manufacture of the pipe were required to build presses of magnitude adapted to this size of pipe and to the proof-test provided for. At one time (1903) the rate of progress in the manufacture of the pipe was so slow that the city was on the eve of going abroad to buy part of the work from the pipe-foundries of Great Britain and Belgium; and it was a pleasure to state that the English pipe-foundries which were called upon in this matter were entirely willing to undertake to meet the conditions of the contract, which were more exacting than any previously laid down in the United States in work of this kind. In addition to the hydrostatic test at the foundry and in the pipe-trench, it was required that there should be cast from the ladle for each piece of pipe at least one tensile test-bar and two cross-breaking bars. The tensile bars were 11 inches long, and approximately 1 inch square, corrections being made for exact dimensions upon breaking the bar in the testing-machine. The cross-breaking bars were 26 inches long, and approximately 2 inches wide and 1 inch deep, and were broken by placing them flatwise on supports 24 inches apart, and applying the load at the centre. The conditions with reference to metal were that the tensile Mr. J. W. Hill.

Mr. J. W. Hill. strength should be not less than 22,500 lbs. (10 tons) per square inch, and that the cross-breaking bars of standard dimensions and span should take a load of 2,280 lbs., equivalent to a maximum fibre-stress of 41,000 lbs. (18·3 tons) per square inch of section, and show a centre deflection of not less than 0·34 inch before breaking. When tenders were received for this contract it was found that the difference between the cost of constructing the work with cast-iron pipe and with riveted steel pipe amounted to about $186,000·00 (£38,130), an amount which in his opinion was too small to justify the risk incidental to laying in city streets these large lines of parallel pipes of material of less known durability than cast-iron; and it was gratifying to note that the tests of the pipe at the foundry and after laying in the trench had never developed a fracture. Out of more than 2,500 pieces of this pipe manufactured to the present time, not one had broken in the proving-press under pressures sometimes as high as 350 lbs. per square inch; and naturally none had broken in the trench under the lower pressure of 200 lbs. per square inch, although the latter might happen should the pipe be injured in transportation. In centering these pipes, instead of following the usual practice of centering from the outside, after the first few lengths were laid this was done entirely from the inside; an inspector inside the pipe indicating by taps of a hammer on the barrel the direction in which the spigot was to be wedged in the socket to bring about concentricity of the spigot end of one pipe and the socket of the next. In the placing of the spigot of one pipe in the socket of the next, an allowance of $\frac{1}{8}$ inch was made for longitudinal motion, partly to compensate for such temperature-changes as might take place in the length of the line before the trench was filled, and partly to allow for inequality in settlement of the pipe after filling, which would not be so well provided for if the spigot of one pipe was brought solidly home against the shoulder of the hub of the next pipe. Valve-chambers were provided at intervals of about 4,000 feet, so arranged with line- and branch-pipe valves as to permit of cutting out of service temporarily any line of pipe which might be injured, and confining the supply during the period of repair to the remaining lines of the system. The worst condition that could arise in service would be the temporary loss of the use of one line of pipe between two valve-chambers, the remaining lines between the two chambers still being serviceable; while of course all the three lines—or four lines when completed—beyond the chambers nearest to the point of rupture would be unaffected by the break. Where the pipe entered the valve-chamber

it was reduced from 60 inches to 48 inches in diameter, partly because the manufacture of 60-inch stop-valves had not been as extensive, nor the use of valves of such size as well established, as those of 48-inch stop-valves, and partly because the height of such stop-valves would have materially increased the depth of the trench in which the pipe was laid. The number of chambers, and therefore of reductions in size from 60 inches to 48 inches in each line of pipe between the pumping-station and the terminal point at Torresdale and Kensington Avenues, was six, and the estimated loss of head by reason of the use of reducers from 60 inches to 48 inches, of 48-inch tees and crosses in the chambers, and of 48-inch stop-valves at each chamber, as computed by the formula given by Weisbach,[1] was less than 0·35 foot for a velocity of 7·39 feet per second through the 48-inch stop-valves and special castings, corresponding with a velocity through 60-inch pipe of 4·73 feet per second: an amount altogether too small to raise any doubt of the wisdom of the reduction of the diameter of pipe from 60 inches to 48 inches while passing through the valve-chambers, in view of the very large reduction in the cost of trenching by the use of 48-inch stop-valves. All valve-chambers were constructed with concrete floors and side and end walls. Upon the walls were placed rolled-steel beams, with arches sprung from the lower flanges of the beams, to support the roadway and paving which was carried over the chamber. Access to the chambers was provided through two manholes at corners diagonally opposite. The special castings in the valve-chambers, tees, crosses and taper-pipes, were made of open-hearth cast-steel, having a tensile strength of approximately 60,000 lbs. (26·8 tons) per square inch of section, and the bodies of the 48-inch stop-valves, which were subjected to twisting and oblique stresses during test in the line and in service, were also steel castings, to reduce the chances of rupture. While the lines of pipe were tested to 200 lbs. pressure per square inch upon completion in the trenches, the working-pressure was limited to 125 lbs. per square inch by a series of 4-inch Crosby relief-valves, one of which was placed on each line of pipe where it entered a valve-chamber. In addition to the relief-valves on the pipe-line, each of the pumping-engines (of which there were twelve of 20,000,000 U.S. gallons daily capacity, or a total of 240,000,000 gallons) delivering into these lines of pipe was provided with a 12-inch relief-valve set to lift at 120 lbs. per square inch. Thus the probability of ever producing a water-ram sufficient to injure Mr. J. W. Hill.

[1] J. Weisbach, "A Manual of the Mechanics of Engineering, etc." vol. i. p. 872. London, 1877.

Mr. J. W. Hill. the pipe in service was extremely remote; but anticipating that rupture might occur under unaccountable conditions, and that with the large volume of water flowing through these lines of pipe, great damage might be done to the bottom of the trench, and probably to the parallel lines of pipe, the pipes were everywhere laid upon a concrete floor 9 inches thick, extending from side to side of the trench. It was thought that, if a rupture should occur in service, the washing out of the filling over a pipe would simply open the street and cut the banks at the side, but could not by any chance shift from their positions and injure the adjoining lines of pipe. In constructing the floor, which was laid after the pipe-lines were blocked in position and tested, a concrete cheek, formed as a monolith with the floor and extending up on the curve of the pipe for a vertical height of about 9 inches above the floor, was provided on each side, thus forming a cradle to guard against the lateral movement of the other pipes should one of the lines be fractured in service. In laying this pipe the contractors were required to construct a standard-gauge railway upon the side of the street, and to place thereon a travelling crane with a boom having a reach of 20 feet at right angles to the line of trench, and a capacity, with this reach, of about 10 tons. Each piece of cast-iron pipe weighed approximately 14,000 lbs. ($6\frac{1}{4}$ tons), and each of the 48-inch stop-valves weighed about 22,000 lbs. (9·8 tons), these representing the heaviest castings used in the work. The contract provided that cutting pipes for closures should not be done in the usual manner with a hammer and chisel, but with a machine which would cut the pipe at right angles to its axis, and leave the end of the casting true and smooth, and which would accomplish the work without the possibility of fracture of either section of the cut pipe. In planning this system of pipes it was thought that in time there would be considerable tuberculation of the interior from the iron and lime salts found in the water of the Delaware River, which would be the source of supply for Lardner's Point pumping-station; and manholes were provided in each of the 60-inch by 48-inch taper-pipes through which access to the lines of pipe between chambers could readily be obtained, for the purpose of removing incrustations, and scraping and recoating the pipe. The removal, subsequent to preparing the contract, of a line of 30-inch pipe which interfered with the placing of the first four lines along Robbins Street, indicated that 27 years' use had produced very little deposit and no tuberculation in the pipe, and the manholes which were intended to be placed midway between the valve-chambers were accordingly omitted, although those in the

taper-pipes were retained. Should occasion arise to examine the interior of these lines of pipe and remove obstructions, this could be accomplished by taking out of service one section of the pipe-lines at a time, between any two consecutive valve-chambers. These pipe-lines, with a calculated capacity of 240,000,000 U.S. gallons per day of 24 hours, were pumping-mains intended to convey filtered Delaware River water to several districts of the City of Philadelphia, which at the present time contained a population of about 1,100,000. The filtration of the Delaware water would be accomplished by sixty-five covered plain sand-filters each $\frac{3}{4}$ acre in area, constructed on the banks of the Delaware River, about 3 miles up-stream from the Lardner's Point pumping-station. In the valve-chambers each line of pipe was provided with a $2\frac{1}{2}$-inch relief-valve to be blown by hand. As the vertical undulations in the lines of pipe were not very great between their origin at Lardner's Point pumping-station and the terminus at Kensington Avenue, it was thought preferable to have these air-valves blown from day to day by an inspector, rather than to trust to automatic valves which might fail to act and probably subject the pipe-line to undesirable shocks by reason of accumulated air. At two points it was necessary to carry the pipes over creeks or large sewers, which necessitated the use of flanged pipe and the building of concrete chambers in the line of the sewer or watercourse, to confine the channels to definite widths. Two lengths of flanged pipe were bolted together in the centre of the span lengths, and acted as tubular horizontal girders of 16 feet clear span, of 60 inches internal diameter and 63·4 inches external diameter, calculated with a factor of safety of about 20, for a centre dead load of pipe, water, earth filling and pavement, of 47,256 lbs. (22 tons), and a maximum centre live load of 40,000 lbs. (17·8 tons). An interesting feature of this contract had been the necessity of moving under pressure a line of 48-inch cast-iron pipe about 1,194·45 feet long, which was one of the two pumping-mains from the original pumping-station (Lardner's Point No. 1). To place this pipe in its new position to accommodate the four lines of 60-inch pipe in Tacony Street required that it should be changed 11·5 feet in alignment, and 13·17 feet in level at its lowest point. In carrying out the work the deflection from the old elevation to the new was made on the first 200 feet of the line at each end, the remaining portion being run on a tangent. As about 30,000,000 U.S. gallons of water were being delivered daily through the 48-inch main, it being the principal pumping-main, it was impossible to take Mr. J. W. Hill.

Mr. J. W. Hill. the line out of service, even for a short time, and careful preparations were made for moving it into its new situation while under pressure, and without stopping the pumping-engines. The centre-line of the main in its original position was 1·17 foot shorter than the calculated centre-line for its new position. After it was moved careful measurements showed that the actual draw of the joints had only been 0·93 foot, or 0·24 foot less than was expected. This was very evenly distributed throughout the one hundred joints, and the average movement of the pipe in each joint was slightly more than 0·11 inch. In order to guard against excessive pull in the joints, each pipe was marked before being moved, on the top and on each side; but owing to the fact that some of the pipes rotated 100 degrees, it was impossible to make proper reductions of the plus and minus readings. Small gauges were, however, used continually on these marks, and the lead rings in some cases were drawn out ½ inch or more; but these joints were not disturbed unless a leak occurred, when they were immediately recaulked. Before moving the pipe, the trench for its new position was excavated to line and gradient; excavations were then made under the pipe on about 200-foot sections, and the pipe was gradually lowered to its new gradient, being meanwhile thoroughly braced in position to prevent lateral movement. The greatest depression was reserved until the last, thus allowing any gain to work up to this point. After the pipe had been lowered to the new gradient, wooden skids, upon which iron strips were fastened, were placed beneath each length, and the pipe was then moved laterally into position with screw-jacks. To facilitate moving the pipe the iron strips were kept well greased. The time occupied in moving the pipe was about 1 month, and except for a few hours, when a cracked pipe was discovered, the line was never taken out of service, and was under a uniform pressure of 70 lbs. per square inch. The cracked pipe had probably existed since the main was originally laid, and to avoid delay steel bands were placed around it and tightened up. In the subsequent operations this length of pipe gave no further trouble. To avoid accidents and delays, rigid inspection was maintained both day and night, and men within easy hailing-distance were placed along the line to ensure the immediate closing of the valves at either end in case of accident. After the pipe had been relocated it was allowed to rest for a few days until it had assumed its final position, and all joints were then thoroughly recaulked. The Author's remarks on leakage were not quite clear in the distinction between leakage at the

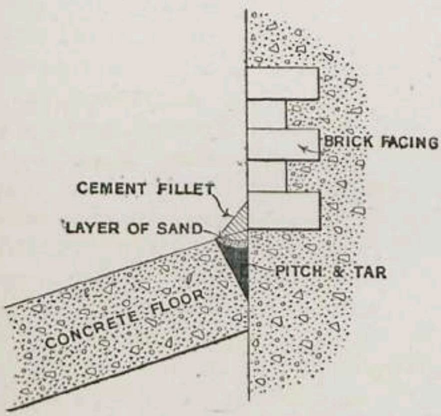
longitudinal locking-bar joints, and leakage at circular lead joints. The Author stated that the lead joints were tight, but examination of locking-bar joints made in the United States had shown them to be water-tight under considerable pressure; so that the leakage referred to by the Author would seem to come from the lead joints where two sections of the pipe were brought together in the double bell. Mr. Hill's experience at Philadelphia had demonstrated that it was almost impossible with cast-iron water-pipe and circular joints to have absolute water-tightness; and that in a long line under a pressure of, say, 100 lbs. per square inch there was a measureable leakage. He would like to know the results of the Author's tests in this respect. Mr. J. W. Hill.

Mr. A. E. Hurse considered that, provided the material used as ballast was of a hard, compact, and dense nature, and sufficient sand was used to fill the interstices, no object was gained by rigid adhesion to a fixed gauge in mixing concrete. Professor C. E. Sherman's experiments on the presence of clay in sand were well worthy of study, and it would be useful if the resistance to crushing of blocks of concrete, made with unwashed and with washed ballast or gravels of different compositions, could be investigated over a period of, say, 3 years, so as to decide this doubtful point. In his own experience on the construction of a covered service-reservoir of 5,000,000 gallons, built of concrete, the use of unwashed pit-gravel had been necessary on account of local difficulties: first a threatened action at law for the silting-up of the watercourses caused by the washing of large quantities of "terrace-gravel" excavated on the site, and secondly the distance and altitude of cartage of materials. The sand screened from the pit-ballast subsequently used was found to contain 8 per cent. of clay, and the aggregate was of good angular shape. The proportions of the concrete were 5 to 1, and there being sufficient sand in the ballast, none was added. During the construction of the walls with the concrete of unwashed ballast, sample masses were cut out at various points; these were found to be well set even after only a fortnight had elapsed since mixing and laying. He regretted that he had no record of the resistance to crushing, but the good quality of the concrete was also shown by the fact that flat arches of about 13 feet span and 13 inches rise, and 12 inches in thickness, were sufficiently set for the centres to be withdrawn after 14 days. Slight cracks occurred at the haunches, but these were cemented up and caused no trouble. In the first coat of rendering unwashed Leighton Buzzard sand, containing $2\frac{1}{2}$ per cent. of impurities, was used, with no undesirable effects. Close Mr. Hurse.

Mr. Hurse. observation of this reservoir for 4 weeks after filling had indicated very satisfactory results. The method of caulking cracks in the floor of a reservoir described at p. 48 was similar to a plan which had been adopted in the floor of a tank of 1,500,000 gallons capacity for a crack resulting from settlement, as illustrated in *Fig. 38.*

Mr. List. Mr. G. H. List was of opinion that the salt-impregnated soil in which the thin steel pipe-line was laid would undoubtedly act injuriously on the metal and greatly shorten the life of the pipes. He had had large experience of pressed-steel sleepers in similar soil, and their life was very short. No doubt the coating described would act beneficially if perfect, but his experience was that in the handling of such pipes the best coating got damaged and allowed corrosion to start. The joints would be the most dangerous places, as here the coating had to be removed to ensure a good lead joint. Paying the joints with hot tar-mixture was sure to be carried out imperfectly in many cases, and so corrosion would get a start. The plan of wrapping and re-dipping, which was being followed at Dudley Port for gas-mains, would protect the body of the pipe, but the wrapping must be cut away for some distance at each end to allow the collar to be slipped on and the lead joint to be run and caulked, so that matters would be no better than before. If it were possible at moderate cost to surround the pipes as a whole with some impervious material, then corrosion would be prevented. He would suggest covering the pipes, after laying and jointing, with Callender bituminous sheeting, or preferably Portland-cement grout, thus protecting them as the lining of "tube" tunnels was protected. This might be done by means of a movable casing halved over the pipes, lined with thin parting-paper, cement grout being run in to give a coating about 1 inch thick. The machine designed and used successfully for caulking the joints was most ingenious; but it was bulky and doubtless somewhat awkward to use in the

Fig. 38.



confined space of a pipe-trench, and could only be used on collar joints with a double face. On socket joints such a machine would have to be firmly clamped round the pipe to enable it to press up the lead joint. On riveted pipes it could not be used, and even on the pipes described a certain amount of hand-caulking was necessary. He had just completed a big pipe-laying job, and at one time he had had difficulties about setting up the joints by hand. To overcome the trouble he had suggested the use of pneumatic caulking-tools—a small portable compressor on wheels, driven by an oil- or steam-engine, with sufficient length of hose to command a definite length of track each side, and ordinary long-stroke riveting hammers working on suitable sets. Owing to questions of first cost near the end of the job, and to the men becoming more reasonable, the proposal had gone no farther; but he thought it would have answered well, and would have been simpler to handle than the machine described. His experience agreed with that of the Author regarding the effect of a proportion of loam or clay on mortar. Much expense and delay had been caused recently on a work with which Mr. List was connected by having to go a long distance for sand, which was no whit better in effect than a sand close at hand which had a small proportion of loam in it. The best brickwork he had ever seen or ever done was on the Rapti Bridge near Gorakhpur on the Bengal and North-Western Railway, India. The mortar used was made of kunkur (slow-setting hydraulic) lime (2 parts) burned with wood in a running kiln, drawn hot, and put into a pan-mill with small native bricks (1 part) taken from old ruins. These bricks had been set in mud mortar, and they were not cleaned or selected in any way. Sufficient water was added, and the mixture was ground for 30 minutes and taken to the work. In 10 days it set so hard that it became monolithic in character, and when any portion had to be dismantled it could only be picked to pieces with a pickaxe or crowbar, the mortar being actually harder than the best "first-class" bricks. He presumed that there was no local limestone available on the Coolgardie scheme; but he thought much more might be done than was now done with the old-fashioned lime mortars, at much less cost, and with quite as good results as with Portland cement. Personally, in cases where work was neither under water nor required to set rapidly for tidal work, he would prefer to use good lime mortar in brickwork and for concrete. He was sorry to say that even in India, where cement was very expensive, and lime was both good and cheap, the tendency was to prefer cement. He knew of one instance where, with both limestone and coal on the spot, cement

Mr. List.

Mr. List. had been used at enormous cost, though he was certain that in the bulk of the work the indigenous materials lime and surki might have been used with safety and economy. The reason was not far to seek: it was unwillingness to take a little trouble, when anything else could be found ready to hand.

Mr. Marsh. Mr. CHAS. F. MARSH remarked that it appeared from the description given in the Paper that only one thickness of caulking-roller was used for each joint. It would be interesting to know whether any trials were made varying the thickness of the tools, as was done with hand-caulking. It might have been better, at any rate, to pass a chisel-edged roller round first, to ease the lead from the barrel of the pipe, before beginning to set up. The average rate of progress for each pipe-laying gang, for the 3-months' period, appeared to have been about thirty-eight pipes per 8-hour day, which was very good. It would be interesting to know what was the record for 1 day. The illustration of the lead-melting and joint-running apparatus did not give a very clear idea of the method of working, and it would be useful if the Author would give a further description. Mr. Marsh thought it would have been better to leave the coating on the end 6 inches of the pipes, as it would not materially affect the running of the joint, and in any case, with an 8-inch collar, 6 inches of scraping at each end of the pipe appeared to be too much. The recoating of pipes in position was never very satisfactory, and it was surely not advisable to destroy the coating more than could be helped. Could the Author state what mixture was used for the necessary repairs to the coating and how it was applied? It was curious that surprise was often expressed at the fact that the presence of small dust in sand used for mortar or concrete increased its strength; for all experimenters who had made tests on the value of sand of different sizes had found that dust improved the sand. Exhaustive experiments carried out by Mr. Feret at Boulogne showed that the resistance of mortars increased not only with the amount of cement used, but also with the combined volume of the cement and sand in a unit volume of mortar. Mr. Feret had found that, when even-grained sands were used, this combined volume became less and less as the grains became smaller, whilst it was larger for sands with varying grains than for those with even grains. For sands with varying grains, the combined volume of cement and sand was larger as the sand-grains became more mixed in size, being greatest when there were no medium-sized grains, and the ratio of large to fine grains was 2 : 1, the cement, of course, being included among the fine grains. By large grains those between $\frac{1}{5}$ and $\frac{1}{12}$ inch were meant; medium

were those between $\frac{1}{12}$ and $\frac{1}{50}$ inch; fine grains were those less Mr. Marsh. than $\frac{1}{50}$ inch. Standard sand was medium-grained, the size of grain varying very little, namely, between $\frac{1}{20}$ and $\frac{1}{30}$ inch. Apparently the presence of a small percentage of clay in sand was not always detrimental, but this must depend on the general character of the sand, and each case should be carefully tested for as long periods as possible, since a sand might give better results for short periods than for long. In a Paper[1] read before the American Cement Users' Association at Indianapolis in January, 1905, Mr. J. C. Hain, Engineer of Masonry Construction to the Chicago, Milwaukee and St. Paul Railway, gave the results of some tests carried out by him on clayey sands, as against standard sands. A sand containing 7·7 per cent. of clayey matter, tested against a standard sand with 3·4 per cent. of clayey matter, gave results 40 per cent. and 30 per cent. higher than the standard at 7 days and 28 days respectively. The difference became slightly less with age, the result of 3-year tests being 20 per cent. higher than the standard. Another sand with 15·7 per cent. of clayey matter, tested against the same standard sand, gave results 10 per cent. higher at 7 days and 26 per cent. higher at 3 years. In another series of tests Mr. Hain made one set of briquettes with unwashed sand containing 6 per cent. of clay, and another set with the same sand after washing. The unwashed sand gave results averaging 25 per cent. higher than the washed sand over periods ranging from 7 days to 2 years. These results bore out those of Professor Sherman, mentioned in the Paper, which showed that sand containing 10 to 15 per cent. of clayey matter gave a mortar approximately 25 per cent. stronger than did clean sand. It would be dangerous, however, to generalize on the somewhat surprising results of these tests, and while it could not be denied that sands which appeared worthless might in reality make stronger mortar than clean sands, it would be extremely rash to use any such sand until careful and extensive tests had been carried out, and its superiority firmly established. If long-period tests could not be waited for, and if for any reason it would be economical to use an apparently dirty sand, it would not be advisable to employ it unless it proved to be at least 5 to 10 per cent. stronger than standard or other selected clean sand at 28 days. The construction of the floor of the Bulla Bulling reservoir in two layers would doubtless prevent cracks from extending right through, but if it had been formed in one layer having a

---

[1] See *Engineering News*, vol. liii, p. 127. Also Minutes of Proceedings Inst. C.E., vol. clxi, p. 396.

Mr. Marsh. grillage of wire of a smaller mesh than that used, placed near the upper surface, Mr. Marsh believed the cracking would have been prevented, since it had been found that such reinforcement prevented the formation of cracks. It would be interesting to know why barbed wire had been used for the grillages. If it had been adopted with the idea that the barbs were necessary to give sufficient hold to the concrete, such a precaution was entirely unnecessary, as plain wires would have given all the hold required. He did not think that wire in the form of a cable formed a good reinforcement, since the concrete could not properly surround the strands and protect them against corrosion; and he believed that a closer mesh of single-strand wires would have been preferable. With the exception of those at the face, the wires in the vertical walls appeared to be unnecessary, since the walls were apparently not designed as being reinforced: in any case the wires in the centre would not be of much use.

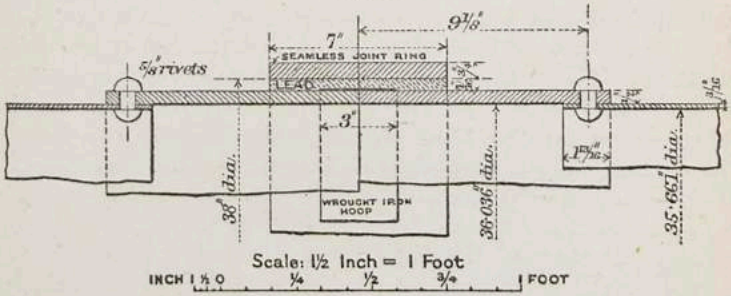
Mr. Moncrieff. Mr. ALEX. B. MONCRIEFF considered that the Institution was indebted to the Author for a lucid and comprehensive Paper on the important Coolgardie water-supply, and personally he desired to thank him for an opportunity afforded him for examining the works, which had been finished in a workmanlike manner and without unnecessary ornament of any kind. Deep regret was naturally felt by the members of the profession in Australia that Mr. C. Y. O'Connor did not live to see his great work completed. Mr. Moncrieff's own experience indicated that the high price at which water was now sold in Western Australia could be maintained only under extraordinary conditions. In the other Australian States it was generally lower, while in South Australia the price of water from the principal supplies fell as low as 6*d.* per thousand gallons. The price which could be obtained for water governed the amount of capital which it was possible to spend in the construction of the works, and it was important to note that from the Coolgardie waterworks there was practically no reticulation, the scheme consisting of headworks, pumping-plant, and leading main only. This might have a vital effect on the ultimate revenue-producing power of the scheme. The Author had touched upon a burning difficulty confronting engineers in Australia when dealing with the question of gaugings of river-flow and rainfall. In that country there were practically no records extending over sufficient time to be of positive value in preparing schemes for large works, and greater responsibility than would otherwise obtain was thrown upon engineers in designing structures and calculating supply. The difficulty was such that it had to be experienced before it could be appreciated thoroughly. He looked upon the Paper as a

Mr. Moncrieff.

valuable addition to the literature dealing with water-conservation in the outlying districts of Australia. The authorities of Western Australia had been fortunate in being able to obtain a clean catchment-area for their great reservoir. This had very frequently not been obtainable in Australia; for the catchment-areas having been alienated and occupied before water-conservation was considered, it was beyond the possibilities of the finances of the States to purchase them back; and in constructing large works Mr. Moncrieff had, where possible, adopted the principle of keeping the reservoirs off the main streams, allowing the flood-waters to pass, and only taking supplies into the reservoirs after the main streams had been scoured. In older countries filtration of this water would be demanded, but the method referred to had proved successful so far. There was not one instance of a large filtration-plant having been constructed in Australia, though there was a small scheme at Broken Hill, N.S.W. The character of the water from the Coolgardie catchment-area was generally recognized as good, but it might ultimately be found necessary to filter the supply for domestic purposes. No provision for meeting this contingency seemed to have been made in the location of the works. The concrete dam at Mundaring was of a very substantial character; but in dealing with almost similar circumstances elsewhere Mr. Moncrieff had preferred to provide a by-wash apart from the dam, as allowing of simpler construction and less liability of damage from excessive floods. He had also found it generally desirable to provide a tunnel in solid ground under the flank of the dam, and to have a separate water-tower, rather than to take the outflow-works through the concrete and build the water-tower as part of the main structure. This was a matter for individual judgment; but it had always appeared to him that to break the continuity of the concrete with an outlet-pipe, or to break the contour of the dam with the excrescence necessary for the construction of a water-tower, was liable to lead to rupture of the concrete under the stresses due to expansion and contraction in the very wide range of temperature obtaining in Australia. Referring to the pipe-line, it seemed to him that no engineer comparing riveted steel pipe with the locking-bar pipe adopted could hesitate to decide in favour of the latter. The punishment of the steel in punching, and the innumerable points at which possible leakage might occur, condemned the riveted pipe at once in comparison with the locking-bar pipe, which consisted of four pieces only. *Fig. 39* showed the method adopted in South Australia in constructing and jointing a riveted steel pipe of

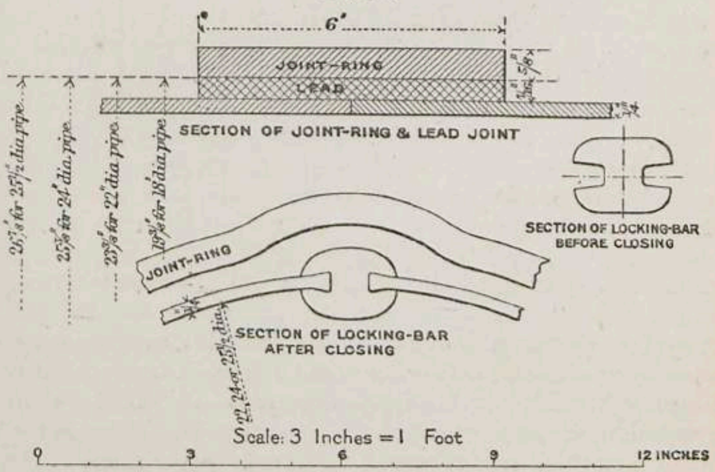
Mr Moncrieff. similar diameter to that used for the Coolgardie works. He had found that the plain ring, allowing for through lead filling, as shown in *Fig. 40*, made an excellent connection for the pipes, and

Fig. 39.



admitted of curves of comparatively small radius being followed in laying such a main, without specials. The difficulty anticipated from the flowing of the lead through the joints of the pipes was entirely overcome by the insertion of a temporary band of iron

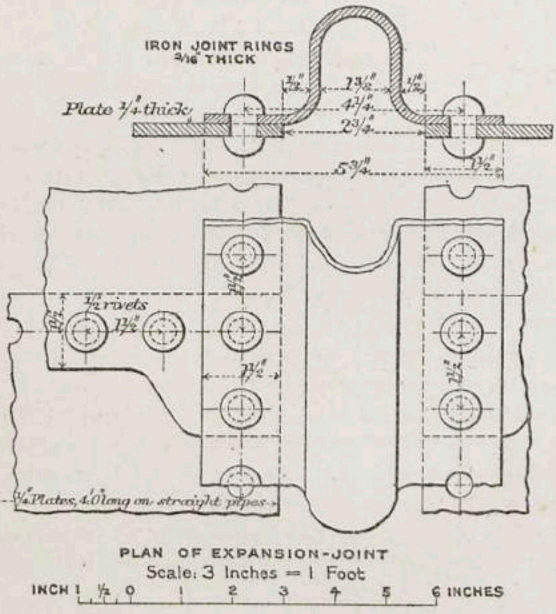
Fig. 40.



expanded in the pipe by means of a screw, and such a joint was certainly not more expensive to caulk than that adopted on the Coolgardie works, while the use of spun yarn—a perishable material—was entirely avoided. With regard to the method of

making the joints, it was generally acknowledged that the Mr. Moncrieff. caulking of the lead round the locking-bar required most care, and as this had had to be done by hand at Coolgardie he failed to see the advantage of caulking the balance of the joint by machinery. The prices quoted for hand- and machine-work seemed high. For laying a similar 26-inch main in South Australia, formed of $\frac{1}{4}$-inch plate closed with locking bars, the cost complete was £4,140 per mile; the cost of laying and jointing being for labour £153 per mile, while labour for caulking only ranged from 2*s*. 6*d*. to 3*s*. per joint. It was possible, however, that higher rates of wages had to

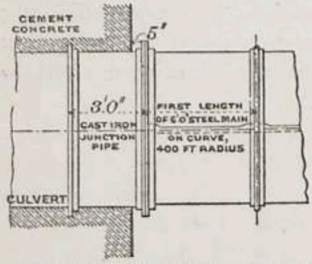
Fig. 41.



be paid in Western Australia. The use of locking-bar pipes, coated in a similar manner to those referred to by the Author, had proved a success in South Australia, in which State the new design of pipe was first adopted by Mr. Moncrieff. The Author had been particularly fortunate in being able to lay his main close to the railway, as carting the pipes over rough country was the common experience, in which case the locking-bar pipe again proved its superiority over the riveted. The methods adopted in laying to prevent the pipes from being injured by exposure in the trenches appeared to have been admirable, and it

Mr. Moncrieff. was satisfactory to note that the simple methods adopted had proved satisfactory without the intervention of the expansion-joint, the introduction of which had been so freely discussed before the work was undertaken. In this connection he had found that for pipes of large diameter the ring joint was very effective where the pipes were exposed to the great variation of temperature obtaining in South Australia. A long length of pipe 6 feet in diameter manufactured as shown in *Figs. 41* and *42*, resting on cast-iron cradles, had remained in this position for many years without the slightest movement, although the temperature to which it was exposed had varied from freezing-point to 170° F. in the sun. The Appendix to the Paper was valuable, but had it been possible to publish more details of the cost, its value would have been considerably enhanced. Perhaps the Author would furnish these details, which would be of much interest to his professional brethren in Australia.

Fig. 42.



JUNCTION OF CULVERT WITH STEEL MAIN
Scale: 1/ Inch = 8 Feet
INS. 12 6 0 1 2 3 4 5 6 7 8 FEET

Mr. Rafter. Mr. George W. Rafter, of Rochester, N.Y., observed that it was unusual to go more than 350 miles for a daily supply of 5,600,000 gallons—or indeed for any quantity; and the combination of gravity and pumping with the 2-years'-storage requirement rendered the Coolgardie water-supply a project worthy of the best effort of the engineers employed upon it. While the Paper was valuable as a whole, the most interesting portion of it to Mr. Rafter was that relating to the yield from the catchment of the Helena River. The Author stated that the catchment-area above the reservoir at Mundaring was 569 square miles, and that, on the face of it, the allowance was excessive for 2 years' supply; but Mr. Rafter had found that the calculations in such cases were frequently insufficient, and whether or not a catchment of 569 square miles would supply 5,600,000 gallons daily, apart from leakage, was purely a question of what water ran into the reservoir. This the Author had foreseen; he had made provision accordingly; and the outcome justified his judgment. Great conservative care in dealing with this question was necessary in other regions besides Australia, because catchments yielded very different quantities of water for the same or nearly the same rainfalls. Frequently it was considered that when the rainfall sank

below a certain minimum, the yield would be nothing. This minimum varied for different regions and conditions. In the case of the Helena River in 1902, the minimum rainfall from which the yield would be nothing was apparently somewhat less than 19·3 inches—on the supposition that the mean rainfall at Mundaring and at York truly represented that of the entire catchment-area, although there was perhaps some doubt on this point, as stated in the Paper. The ratio of yield to rainfall in that year was only 0·2 per cent., or the depth of off-flow on the catchment was 0·039 inch. In 1897 a rainfall of 24·5 inches gave a ratio of yield to rainfall of 0·34 per cent. (0·083 inch), while in 1900 a mean rainfall of 33·25 inches gave a ratio of 3·5 per cent. (1·6 inch). This was a small quantity for a rainfall of more than 33 inches, from the point of view of one familiar with yields in the United States, where, in the eastern part of the country, for this rainfall yields of about 8 to 15 inches might be expected. Apart from the evaporation observations given in Table IV. (p. 51), the preceding statement alone would indicate that evaporation from the Helena catchment-area was large. The gaugings showed that in 1897 and 1902 the ground-water off-flow of the Helena River must have been little or nothing. Mr. Rafter considered that there was a law, though a somewhat obscure one, governing the yield from catchments in all parts of the world; but this law was not yet very precisely determined, nor was it likely to be, because the absorptive nature of the soil of catchment-areas, evaporation, and other conditions differed so much, that every catchment-area was, so far as its yield as a whole was concerned, a law unto itself; and only when all, or nearly all, the conditions were known would it be possible to deduce a formula expressing accurately the relation between rainfall and yield. Nevertheless, there were some general principles which might be applied, although not admitting of much precision. For example, for the Genesee, Hudson and Croton rivers in the State of New York, the following data were available.[1]

Mr. Rafter.

Genesee River.	Inches. Rainfall.	Inches. Off-flow.
Yearly average	40·3	14·20
1895 (minimum year)	31·0	6·67
Difference	9·3	7·53

[1] Much information on the subject of the relation between rainfall and the yield of catchment-areas, including the data here given, will be found in Mr. Rafter's work "Hydrology of the State of New York." Albany, N.Y., 1905. [*Bulletin No. 85 of the New York State Museum.*]—Sec. Inst. C.E.

Mr. Rafter.

	Inches. Rainfall.	Inches. Off-flow.
1894 (maximum year)	47·79	19·38
Yearly average	40·30	14·20
Difference	7·49	5·18
Comparing the maximum year with the minimum—		
1894	47·79	19·38
1895	31·00	6·67
Difference	16·79	12·71
Comparing the storage-periods—		
Average of storage-period	19·40	10·50
Storage-period of 1895 (minimum)	13·20	5·63
Difference	6·20	4·87
Storage-period of 1894 (maximum)	27·71	15·73
Average of storage-period	19·40	10·50
Difference	8·31	5·23
Comparing any two years—		
1892	41·69	15·42
1896	40·68	12·80
Difference	1·01	2·62
Comparing storage-periods—		
Storage-period of 1892	19·84	9·38
Storage-period of 1896	17·84	9·25
Difference	2·00	0·13
Comparing growing-periods—		
Growing-period of 1892	15·30	4·90
Growing-period of 1896	10·28	0·83
Difference	5·02	4·07
Comparing replenishing-periods—		
Replenishing-period of 1896	12·56	2·72
Replenishing-period of 1892	6·55	1·14
Difference	6·01	1·58

Hudson River.

	Rainfall.	Off-flow.
Yearly average	44·21	23·27
1895 (minimum year)	36·67	17·46
Difference	7·54	5·81
1892 (maximum year)	53·87	33·08
Yearly average	44·21	23·27
Difference	9·66	9·81

Mr. Rafter.

	Inches. Rainfall.	Inches. Off-flow.
Comparing maximum year with the minimum—		
1892	53·87	33·08
1895	36·67	17·46
Difference	17·20	15·62
Comparing the storage-periods—		
Average of storage-period	20·62	16·10
Storage-period of 1895 (minimum)	15·79	11·68
Difference	4·83	4·42
Storage-period of 1892 (maximum)	24·95	22·50
Average of storage-period	20·62	16·10
Difference	4·33	6·40
Comparing any two years—		
1898	48·28	27·12
1891	42·96	20·56
Difference	5·32	6·56
Storage-period of 1898	22·80	16·81
Storage-period of 1891	20·69	16·59
Difference	2·11	0·22
Growing-period of 1898	13·52	3·24
Growing-period of 1891	13·49	2·07
Difference	0·03	1·17
Replenishing-period of 1898	12·19	5·27
Replenishing-period of 1891	8·78	1·90
Difference	3·41	3·37
Croton River.		
Average of storage-period	23·68	16·83
Storage-period of 1897	20·55	14·64
Difference	3·13	2·19
Storage-period of 1898	28·81	20·08
Average of storage-period	23·68	16·83
Difference	5·13	3·25
Comparing any two storage-periods—		
Storage-period of 1896	24·84	18·01
Storage-period of 1895	19·55	14·78
Difference	5·29	3·23
Storage-period of 1888	30·33	21·74
Storage-period of 1883	19·03	11·37
Difference	11·30	10·37

Mr. Rafter. NOTES.—*Genesee River.*—This, and one of its tributaries, Oatka Creek, were gauged, with the exception of a certain period, from 1890 to 1898, inclusive. The sequence of these gaugings was as follows: from April, 1890, to November, 1892, gaugings of Oatka Creek (catchment 27·5 square miles) were made; from December, 1892, to August, 1893, inclusive, gaugings were not kept of either Oatka Creek or Genesee River, and for this period the yield is computed approximately from the rainfall by an application of the principle indicated in the foregoing tabulation; September, 1893, to February, 1897, inclusive, the record is that of Genesee River at Mount Morris (catchment, 1,070 square miles); in March, 1897, the dam at Mount Morris over which the gaugings were made was carried away by a flood and the record for the balance of that year and for the year 1898 is deduced from the record at Rochester, where gaugings have been kept since 1893. The catchment-area at Rochester is taken at 2,365 square miles.

Hudson River.—In October, 1887, daily measurements of the flow of Hudson River were begun and have continued every working-day from that time to the present. These measurements are made at the dam of the Duncan Company, at Mechanicville, where this company works a paper-mill, using the entire flow of the river from a catchment-area of 4,500 square miles. In order to obtain the complete flows, a record has also been kept of the number, size, kind and discharge of turbine water-wheels in use during the same period. The flow of Sundays and holidays, when no observations were taken, has been assumed as a mean between the preceding Saturday and the following Monday. The detailed Table of Hudson River yield, from which the foregoing tabulation is drawn, is for the 14 years 1888–1901, inclusive.

Croton River (catchment, 339 square miles).—This is appropriated as the water-supply of the City of New York. Records of the yield have been kept from 1868 to the present time, but the foregoing statements include only the period 1877–99, inclusive.

The statements are made with reference to a "water year" extending from December to November, inclusive, and divided into a storage-period (December to May), a growing-period (June to August), and a replenishing-period (September to November).

On the Genesee catchment, an average of 10·5 inches runs off in the storage-period, 1·7 inch in the growing-period, and 2·0 inches in the replenishing-period. On the Hudson, an average of 16·10 inches runs off in the storage-period, 3·45 inches in the growing-period, and 3·72 inches in the replenishing-period. On the Croton, an average of 16·83 inches runs off in the storage-period, 2·57 inches in the growing-period, and 3·43 inches in the replenishing-period. The average rainfalls corresponding to these yields can be obtained by examining the tabulation. The Genesee River is distant from the Hudson and the Croton about 240 miles.

The foregoing figures related to rivers in the eastern part of the United States, where annual rainfalls ranged from a minimum of less than 20 inches to maxima of over 60 inches. The rainfall and yield of the Helena River might be contrasted with those of the Loup River in Nebraska. The catchment-area of the Loup River was 13,542 square miles. It issued from a sloping, gently rolling country, with a soil consisting largely of porous sand, into which a portion of the rainfall sank, finally appearing as ground-water flow into the stream. The river was 250 miles in length,

elevated 1,500 feet above sea-level at its mouth and about 3,400 feet at its source. The average length of the catchment-area was 230 miles and its width 59 miles. The following were the rainfalls for the years 1891–98, inclusive, as kept at seven somewhat irregularly distributed stations in or near the Loup River catchment-area. The elevations of the stations above sea-level were given under the names, in feet. Mr. Rafter.

Year.	Ainsley. (2,307.)	Bassett. (2,323.)	Burwell. (2,180.)	Lexington. (2,385.)	North Loup (1,967.)	North Platte. (2,841.)	Ravenna. 2,008.)	Average of Seven Stations
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
1891	30·42	24·75	25·20	37·46	30·96	23·36	35·92	29·73
1892	25·36	24·46	21·85	26·71	22·56	20·37	25·60	23·70
1893	15·88	15·74	15·69	24·79	15·46	13·16	18·13	16·98
1894	10·81	12·40	12·77	15·37	14·17	11·21	15·67	13·20
1895	20·94	18·32	20·22	23·94	22·88	14·58	20·26	20·16
1896	21·32	20·07	20·71	30·03	30·37	16·52	27·50	23·79
1897	27·13	25·86	17·70	30·85	28·10	17·09	32·75	25·64
1898	17·86	17·78	18·18	22·80	16·50	15·54	18·50	18·17
Mean	21·21	19·92	19·04	26·49	22·62	16·48	24·30	21·42

The following Table gave the approximate yield, in inches on the catchment-area, of Loup River at its mouth for certain months for the years 1895–1901, inclusive. The yield of this stream was stated to be, within limits, uniform from month to month, and it was on this basis that the column, "Proportionate quantity per year," had been computed.

Year.	April.	May.	June.	July.	August.	September.	October.	November.	Measured Quantity for Indicated Months.	Proportionate Quantity per Year.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
1895	0·23	0·25	0·30	0·18	0·19	0·20	0·21	0·25	1·81	2·72
1896	0·34	0·25	0·25	0·23	0·22	0·20	0·23	..	1·72	2·95
1897	0·30	0·21	0·20	0·22	0·15	0·16	0·36	..	1·60	2·74
1898	0·23	0·30	0·33	0·17	0·23	0·17	0·21	0·22	1·86	2·79
1899	0·27	0·26	0·32	0·18	0·20	0·18	0·16	..	1·67	2·86
1900	0·28	0·29	0·29	0·28	0·29	0·28	0·23	0·23	2·18	3·27
1901	0·35	0·39	0·35	0·24	..	..	..	..	1·33	3·99
Mean	0·29	0·28	0·29	0·21	0·21	0·20	0·23	0·23	1·74	3·05[1]

[1] Approximate.

The foregoing rainfalls and yields of Loup River indicated that the minimum rainfall which might occur without any ac-

Mr. Rafter. companying yield was considerably less than in Australia, and might be not more than 8 to 10 inches. This difference was probably due largely to difference in evaporation, absorptive capacity of the soil, and other physical conditions, although the evaporation in Nebraska was large. At Lincoln, in 1895, it was 48·4 inches; at Omaha, in the same year, it was 54·6 inches; and at North Platte, 48·6 inches. These evaporations were from a free water-surface, as determined by pan measurement at Lincoln, and computed from the readings of wet- and dry-bulb thermometers at Omaha and North Platte. The velocity of the wind was an important element in evaporation, and the following data on this point referred to Omaha for the calendar year 1895:—Average velocity of wind for the month of May, 10 miles per hour; for the month of July, 6·9 miles per hour; and for the entire year, 8·5 miles per hour. In Southern California, the conditions approximated somewhat to those in Australia in the region discussed by the Author. As illustrating this, some phases of the yield from Sweetwater River, which had a reservoir upon it, with a catchment-area of 186 square miles above the reservoir, might be considered. The following records were for a "water year" from September to August, inclusive, as most nearly agreeing—according to Lippincott in his California Hydrography—with the natural division for this

Year.	Rainfall.	Off-flow.	Evaporation.	Wind.	Temperature of Air.
	Inches.	Inches.	Inches.	Miles per Hr.	°F.
1892–93	20·0	1·61	49·6	4·9	61
1893–94	14·8	0·14	48·7	5·1	60
1894–95	27·1	7·12	46·3	5·2	59
1895–96	19·5	0·101	45·2	5·2	61
1896–97	24·9	0·665	..	5·3	61
1897–98	18·2	0·0005	61·9	5·9	60
1898–99	18·5	0·025	..	5·8	59
1899–00	19·1	..	..	5·7	61
1900–01	23·7	0·087	..	..	62
1901–02	20·3	..	..	..	..

NOTE.—The rainfall is taken as a mean of three stations, as follows:—At two stations in the catchment of Sweetwater River, namely, at the reservoir (elevation 250 feet) and 25 miles east of the reservoir (elevation 3,500 feet), and at a third station (elevation 4,800 feet) 3 miles from the watershed-line between the catchments of the Sweetwater and San Diego rivers. Probably the rainfall of this third station is excessive, and certain corrections have been made which render the deductions somewhat approximate.

region. The Table showed that for the conditions of Sweetwater River in Southern California the limit of rainfall with no yield varied between about 15 inches and 19 or 20 inches, the yield depending upon the distribution of the rainfall, evaporation, absorptive condition of the soil, and other elements. While the preceding illustrations were rather general, nevertheless, taken in conjunction with the Paper, they fairly substantiated the statements that the yield, while roughly proportional to the rainfall, was not in very precise relation thereto; and that the quantity of rainfall required to produce some off-flow varied in proportion to evaporation, physical conditions of the catchment, etc., in different parts of the world. Mr. Rafter.

Mr. Charles W. Smith thought the Coolgardie waterworks might certainly be characterized as one of the boldest schemes ever carried out; and considering the paucity of data regarding rainfall and river-flow at the time of its inception, and the unsatisfactory ratio of yield to rainfall, as ascertained after the commencement of the work, the designers were to be congratulated on the fact that the storage-reservoir had been satisfactorily filled. Engineers with Australian experience, however, and with a knowledge of the variableness of the rainfall in that country, would readily concede the wisdom of providing storage capable of conserving sufficient flood-waters to tide over any protracted period of drought, certainly not less than would suffice for 2 years. The Author anticipated that possibly there might be a flood-discharge over the weir-crest of a depth of 5 feet, but did not state how he arrived at this conclusion. In the absence of accurate information as to the magnitude of floods, what assurance was there that any depth estimated on insufficient data might not be largely increased, as had been Mr. Smith's own experience at the Laanecoorie Weir, on the Loddon River, Victoria? In that case the maximum flood-discharge had been inferred from gaugings made at low states of the river only, and checked by a formula applied to the drainage-area. This had given a maximum volume of $1\frac{1}{4}$ million cubic feet per minute; but actual flood-measurements, made after completion of the weir, had proved it to be as much as $2\frac{1}{4}$ million cubic feet per minute for floods of a few hours' duration. For the passage of the smaller volume only, a depth of 8 feet on the sill had to be reckoned with, and in order that no damage might result from the too sudden discharge of flood-waters over the weir, it had been decided to keep the crest of the masonry 5 feet below full-supply level, and to introduce automatic flood-gates to close the space up to that height. These gates had been designed Mr. Smith.

Mr. Smith. somewhat on the lines of those described by Mr. Chaubart.[1] The installation, which had been in work for about 12 years, was most satisfactory, and had allowed the maximum floods to pass without damage either to weir or to flank-works. The adoption of this system appeared to Mr. Smith to be the wiser course when dealing with weirs of this class, where the flood-waters were of uncertain volume. So large a portion of the capital cost of the Coolgardie scheme had been expended on the pipe-line, and the life of this main depended so much on the value of its protective coating, that a few words as to Mr. Smith's experience of the Sydney water-supply works might be of some interest. There many miles of steel pipes were laid, of both the riveted and locking-bar type, and ranging from 72 inches to 8 inches in diameter. In all the coatings applied Trinidad asphalt had been an invariable ingredient, in proportions ranging from 75 to 50 per cent. In the earlier contracts for these pipes coal-tar had been specified to be used in conjunction with the asphalt in equal parts, but later maltha, in the proportion of 25 to 50 per cent., had been substituted for coal-tar. Speaking generally, none of these mixtures could be pronounced a success. Where pipes were laid above ground, as in the case of 9½ miles of 72-inch pipe (forming duplicate mains between Pipe Head and Potts Hill balancing-reservoir), the exterior coating had proved an absolute failure. One of these mains had been scraped and cleaned, and recoated with cement wash, which also had proved ineffective. Recent experiments with a paint composed of boiled linseed-oil and Portland cement in the proportion of 1 gallon of oil to 5 lbs. of cement, as used by the Public Works Department of Queensland for painting water-tanks both inside and outside, had proved so satisfactory that its extensive use, more especially for outside coatings, had been decided on.

Mr. Strange. Mr. W. L. STRANGE observed that the weir forming the storage-reservoir was one of the highest in existence, while the length of the rising-main and the magnitude of the pumping installation were not equalled in any existing waterworks. The work had been carried out under very difficult conditions in a manner which reflected the utmost credit upon all concerned. The scheme was probably unique in regard to the very small amount of the yield from the catchment—0·20 to 3·50 per cent. of the annual rainfall of 19·3 to 33·25 inches. The originators of the project were to be congratulated on forecasting the amount of the yield so accurately. Apparently the larger part of the catchment-area was very flat as

[1] *Annales des Ponts et Chaussées*, 1855, pt. ii, p. 230.

well as very absorptive. The test of the amount of evaporation, made at the reservoir, indicated that the loss on this account, in ordinary circumstances, should be moderate. The remaining factor of the disposal of the rain-water—percolation—should therefore be high, were it not for the densely wooded nature of the catchment-area, which would reduce it by increasing the evaporation. Probably the true explanation of the small amount of the yield was, as pointed out by the Author, that the rain-gauge stations registered falls greatly in excess of the average amount. The experience gained from the catchment-area confirmed that obtained in other countries, namely, that it was far better to depend for a gathering-ground upon a smaller area with abundant rainfall than upon a larger one with deficient precipitation. With regard to the proposal to supplement the natural catchment-area by diverting the off-flow from part of the neighbouring well-watered country into the reservoir, it must be remembered that catch-water drains could not be economically made of sufficient size to carry the whole of the off-flow from storms, which were the chief sources of replenishment. The advisability of constructing subsidiary reservoirs to feed the main reservoir might therefore be considered. Possibly it would be practicable to increase the yield from the natural catchment by denuding it, partially or wholly, of its timber. This should improve the potable quality of the water, although it might lead to an increase of silt-deposit in the reservoir. It was not clear from the drawings why the weir had been designed with so high an overfall, seeing that this had apparently entailed the thickening of the section of the structure. It would seem from Figs. 6, Plate 1, to have been possible to provide (with or without undersluices, and with a somewhat increased depth of overflow) a waste-weir of sufficient discharging-power and with a low overfall at the south end of the dam. Doubtless, however, this matter had been fully considered by the designer of the work. In any event, the provision of undersluices would have saved the prolonged flow over the weir which apparently would occur in all years of good rainfall. As was so frequently the case, the foundations of the work had proved worse than anticipated; for this reason, when estimating the cost of such structures, it was always advisable to allow a large amount for contingencies. The foundations were very irregular in cross-section, and it was fortunate that this had not led to any cracks due to unequal settlement. The Author did not explain why the outlet was placed at practically the deepest point of the longitudinal section, where also the cross section of the foundations was very irregular, nor why the

Mr. Strange.

L 2

Mr. Strange. lowest outlet-valve was located so near to the base of the weir. The contents of the reservoir at this level were inappreciable, and in a few years' time they would probably be much decreased by silting. By raising the sill, apparently a safer position for the outlet could have been selected, and the pumping-lift would thereby have been diminished. The vertical intervals at which the outlet-pipes had been placed were unusually large, and it would have been an improvement if arrangements had been made to draw off the supply at more numerous points. The permanent object of having a scour pipe at the base of the outlet was not clear, as this would not enable any appreciable amount of silt to be removed from the reservoir-basin, nor could it aid greatly in diminishing the flood-discharge over the weir. The selection of the locking-bar pipe had resulted in much saving, but it was a somewhat bold measure to adopt it on so large a scale with only the limited experience of a very much shorter main as a guide. The Author did not state whether any form of reinforced concrete main had been considered; most likely, at the time, sufficient experience of this form of construction had not been gained. Although it would probably have involved more pumping-stations, so as to reduce the pressure on the main, a pipe of this kind would seem to be peculiarly suited to a line of country heavily charged with corroding salts. It was noteworthy that the long length of the main had exercised a purifying influence on the quality of the water. It was a question, however, if filtration before the water was admitted into the rising-main would not have been advisable, in regard to the population served, as well as lessening the incrustation of the pipes. The "sand cuts" mentioned by the Author were interesting. Presumably the escaping water was partially confined by the filling over the main, and was thus able to give the sand abrading-power. The Author did not say if any permanent arrangement had been made to prevent this damage from occurring: surrounding the pipes with gravel or clinker might be a remedy. Mr. Strange noticed that the total storage-capacity of the reservoirs *en route* and at the end of the main was 28·68 million gallons, or 5 days' supply, of which 12 million gallons, or a little over 2 days' supply, was at the end; this seemed a small allowance for so long a length of main, but it would doubtless be ample as long as the daily consumption remained as low as it was at present.

Mr. Williams. Mr. GEO. B. WILLIAMS thought that, considering the very small margin between the loss from evaporation and absorption and the rainfall, and also the fact that the off-flow depended almost entirely on the few very wet days in the year and on the volume

of water falling on those days, the irregularities in the ratios of the yield to the total annual rainfall were not surprising. If the average percentage were taken for a series of years, a figure could no doubt be obtained which would be approximately correct for any series of years, providing the periods taken in each case were long enough. It would have been more satisfactory if, when dealing with the discharge, some statistics as to the rainfall on the watershed had been available. Unfortunately no rain-gauges appeared to have been fixed, and this question was dealt with in a perfunctory manner. The reasons given by the Author for assuming that the rainfall on the upper portion of the watershed was less than at York hardly appeared conclusive. If the average loss from evaporation and absorption could have been obtained, expressed in inches over the whole watershed, the result would have been of considerable interest. From Table I. (p. 50) and Table III. (p. 51) and from the map (Fig. 2, Plate 1), the rainfall on the watershed below the weir could be estimated; and for the 3 years 1899–1901 the average loss from evaporation and absorption appeared to have been about 31 inches for the larger watershed of 50 square miles, and 26½ inches for the smaller area of 10 square miles. These losses seemed somewhat large when the discharges were compared with those from the catchment-areas of the Nepean and Cataract Rivers near Sydney.[1] For 6 years the average loss amounted to 25½ inches for the Nepean watershed and 28½ inches for the Cataract. It would be supposed that the loss on the Helena watershed would be less, for the rain all fell in the winter months; and it would be reasonable to expect it to be not much more than 20 inches per annum. In the absence of accurate data a nearer estimate than this could not well be obtained, and it might be somewhat wide of the mark. Assuming that 20 inches was the average annual loss, it would be found from Tables I. and III. and from the Table on p. 10 that the average rainfall for the whole watershed for the 25 years, 1876-1901, would have been nearly 21½ inches, which was less than the mean of York and Mundaring, but more than the Author's figure of 18½ inches on p. 9. This latter figure appeared to be the result of pure guess-work. Mr. Williams.

Mr. GEORGE PHIPPS WILLIAMS, having had the pleasure of intimate acquaintance with the late Mr. O'Connor during his career in New Zealand, was glad of an opportunity of expressing his admiration for the boldness and originality displayed throughout the magnificent scheme designed by him and described in the Paper. Mr. Phipps Williams.

[1] Minutes of Proceedings Inst. C.E., vol. lxxv, p. 176.

Mr. Phipps Williams.

Mr. O'Connor's plans exhibited the same breadth of view and the same mastery of detail for which he was noted when on the goldfields of the west coast of New Zealand, where his water-races and other works remained as monuments of his professional skill. The Paper seemed to deal a death-blow to all arbitrary formulas for the discharge from catchment-areas in relation to the rainfall, or indeed to any formula at all. Countries varied so much in local conditions that only the closest possible study of all the factors in each case could give results that were even fairly approximate. In the Canterbury Plains of New Zealand, large areas had no visible off-flow, and some of the rivers had an underground flow greater than that above ground. Some of the former found outlet as springs on the tops of downs and on slopes lying on the reverse side of the watershed from the catchment-area which supplied them. Some of the flow found its outlet at the bottom of the ocean, having been previously partially tapped by the numerous artesian walls in the neighbourhood of Christchurch. In the present case, where only 0·2 per cent. of the rainfall in 1902 was discharged at the weir, evaporation and not percolation seemed to have been the factor to which the enormous loss was ascribed. There was apparently no percolation at the Mundaring reservoir, though it had been anticipated; but it was not clear whether percolation might not cause loss through fissures elsewhere within the catchment-area. Another point raised by the Paper was an important one, affecting all engineering undertakings, especially in the Colonies, where economic development was often rapid. It was the question how far it was economically politic to design works ahead of actual requirements. In water-supply schemes it was no doubt sound hydraulic practice that a main should have the full capacity of the expected ultimate discharge, and circumstances might perhaps warrant a like course for pumping-station buildings; but with regard to the pumping-machinery itself, it was clearly advisable in a tentative scheme of this sort to cut down expenditure until prospects of increased consumption warranted further outlay, provided sufficient reserve power had been allowed for breakdowns. In the present case it appeared that, although the pumping-engines had been erected about 3 years, the consumption of water had never yet exceeded one-fourth of the amount which they were designed to supply. Their total cost was set down as £290,000; so that, including a proportional part of the reserve of power as estimated, the sum of £72,500 would have sufficed to cover the cost of all requirements and contingencies to date. Further, the actual lift was also much less; because, taking the discharge at 1,400,000

Mr. Phipps Williams.

gallons per day, the velocity in a 30-inch pipe would not exceed 0·53 foot per second, which would reduce the frictional head to an almost nominal amount—something less than 3 inches per mile; or taking it at 80 feet for the 308 miles, and allowing 50 per cent. for deterioration of pipes and waste head, it would amount to only 120 feet. The total natural head, including height to be surmounted and losses at reservoirs, was 1,500 feet, and this added to the frictional head as above gave a total head of 1,620 feet only, which was exactly three-fifths of the total estimated head of 2,700 feet for the full discharge of 5,600,000 gallons. Thus the cost of the engine-power required for the present maximum load might be further multiplied by that fraction, making a reduction again from £72,500 to £43,500, including the same proportion of reserve power as had been provided. This sum subtracted from £290,000 gave £246,500 as the amount by which the cost of pumping-machinery exceeded that of present requirements. The interest on this sum at 4 per cent. amounted to £9,860 a year, and assuming the engines to have been erected 3 years, it appeared on the face of it that a sum of nearly £30,000 might have been saved to date on this item alone. No doubt in any undertakings like this the engine-power should be kept well ahead of present requirements, even after allowing ample reserve power; but the margin allowed here seemed excessive, and he gathered from the Paper that this excessive first outlay was not contemplated in Mr. O'Connor's original estimates.

The Author.

The Author, in reply, observed that he had perused the Correspondence with much gratification, and he felt that waterworks-engineers were especially indebted to those who, in addition to comment, had furnished valuable data regarding water-mains and the yield from catchment-areas. He would ask for consideration when dealing with some of the comments, as his hands had not been altogether free in construction, owing to orders for material already placed. There were naturally features of the works in which he himself did not concur; and although he had altogether avoided showing this in the Paper, he might not be able in reply to put the case on some points as favourably as his predecessor would have desired. The restricted demand for water as compared with the original estimate had been dealt with fully in his reply to the Discussion. The initial report on which the Western Australian Parliament sanctioned construction was of a very general character, so that although the money estimate as a whole was intended to be and had proved practically correct, the details could not be expected to be similarly correct. Engineers of experi-

The Author. ence in responsible colonial positions were aware that the large number of reports which an engineer-in-chief had to submit, on proposals of every description, rendered it impossible to conduct a department economically unless initial schemes were kept of a general character; and therefore in reply to Mr. Phipps Williams it was enough to add that, although the allowance for pumping-machinery in the initial proposals was comparatively small, there was no doubt in the Author's opinion that Mr. O'Connor was right in concurring in a larger allowance for friction-head, etc., and a corresponding increase in pumping-machinery. The works had been carried out by departmental labour, not by contract, and designing had proceeded almost *pari passu* with construction, so that alterations had been possible, except of course in cases such as that of the pumping-machinery, for which orders had been placed early. As stated in the Paper, a considerable amount of reticulation had been carried out, domestic and other services having been furnished throughout in the townships of Coolgardie, Boulder, Kalgoorlie, Boulder Mines, Southern Cross and Northam, and those of Midland Junction and Guildford were now in hand. But as the original proposals did not include reticulation, and as there was nothing very special in these works as carried out, except perhaps the use of a positive meter on every service, he had not entered on any description in the Paper. This has evidently misled Mr. Moncrieff, whose remarks also as to the high price of water in Western Australia could be held to apply only to the Coolgardie scheme. Elsewhere in that State prices were lower—in fact, below those of other parts of Australia: for while the water-rates in Melbourne, Sydney and Adelaide were respectively 6*d.*, 7*d.*, and 1*s.* in the pound, and while the measurement-rates per 1,000 gallons at these places were respectively 1*s.*, 1*s.*, and 6*d.* to 1*s.* 3*d.*; on the other hand, the Author had been fortunate in obtaining sanction for the corresponding rates at Fremantle, the principal port of Western Australia, to be reduced to 6*d.* in the pound, and from 1*s.* per 1,000 gallons generally to as low as 4*d.* for certain purposes. Even at these rates there was a handsome return on the capital invested: moreover, in the western State there were several possible irrigation-schemes from which water could be profitably sold at a low figure. It was very interesting to note that the American figures confirmed the small yield possible from catchment-areas in climates such as that of Australia, so different from more humid places. Mr. Geo. B. Williams's statement, that it would have been more satisfactory to obtain statistics as to the rainfall on the watershed before

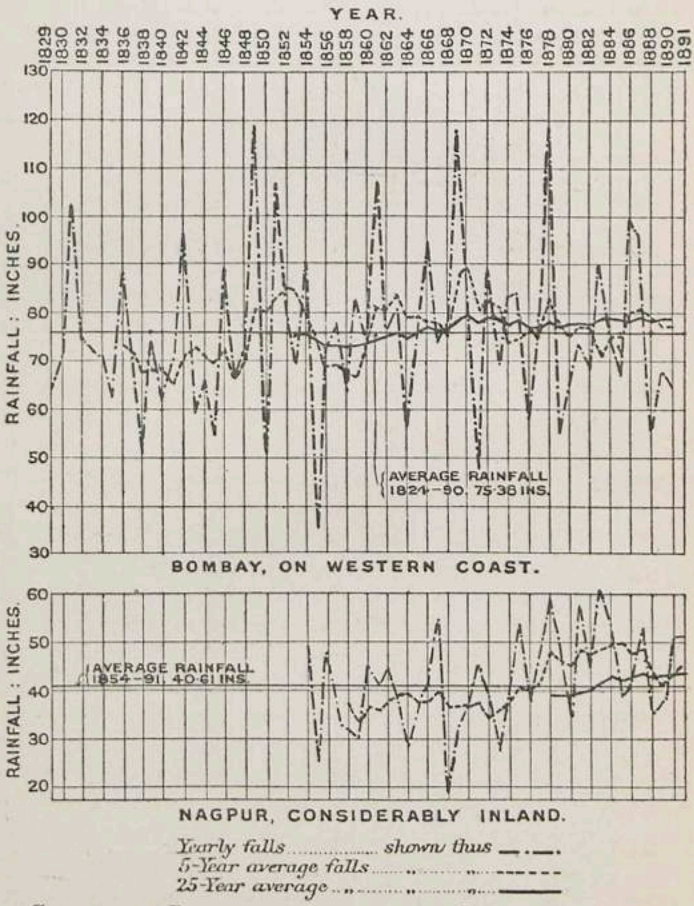
The Author.

commencing work, was no doubt correct in theory when applied to the best-known portions of old countries in the present day; but such procedure in the Colonies would immensely retard development, and would certainly have sadly hampered the great English pioneer engineers of the last century. Although it was out of the question to expect accurate or even regularly correct results by forecasting where long records were non-existent, still some method of estimating was necessary; and the results detailed by Messrs. Folwell, Hering and Rafter seemed to fit the Author's theory better than that of the simple percentage reduction mentioned by Mr. Bruce and referred to hereafter. The point was one of the greatest possible importance, and the Author ventured therefore on the following amplification. The methods of designing on the basis of the yield might be divided into four, namely: (1) when very ample quantities were available and no storage, or very little, was provided, as when a spring or perennial river was tapped; (2) when a whole year's demand was considered and storage was provided accordingly; (3) when a succession of dry years—generally three—were taken as the basis, and the storage was intended to equalize the yield in this period; and (4), the worst case of all, when sufficient reservoir-capacity had to be provided to render the surplus of years of heavy rainfall available for use in dry years over a long period, perhaps 20 years or more. Mr. Fuertes's interesting and valuable contribution regarding minimum yields affected only the conditions detailed under (1) and (2). Of the other gentlemen who had written on this point, all except Messrs. Folwell and Hering appeared to have dealt with it under the comparatively fortunate circumstances detailed under (3), and of course regarding this there were valuable data already extant, as, for instance, Sir Alexander Binnie's Papers on rainfall;[1] but even in the British Isles and other well-watered and humid countries, owing to the increasing demand for domestic, power, and other purposes, the time was coming—if indeed engineers had not already been faced with the question—when the problems must very often be solved under the conditions detailed under (4), and be solved more economically than was possible under those of (3) only. In the Author's experience alone the economies effected by allowance for long-date storage had ranged in certain cases from

[1] "The Nagpur Waterworks: with Observations on the Rainfall, the Flow from the Ground and Evaporation at Nagpur; and on the Fluctuation of Rainfall in India and in other places." Minutes of Proceedings Inst. C.E., vol. xxxix, p. 1.
"On Mean or Average Annual Rainfall, and the Fluctuations to which it is subject." *Ibid*, vol. cix, p. 89.

The Author. £25,000, in one instance, to a 30-per cent. reduction of the cost per 1,000 gallons in another. It was in the hope, therefore, that, as suggested by Mr. Fuertes, other data might be forthcoming later on regarding long-date results, that the Author drew attention to

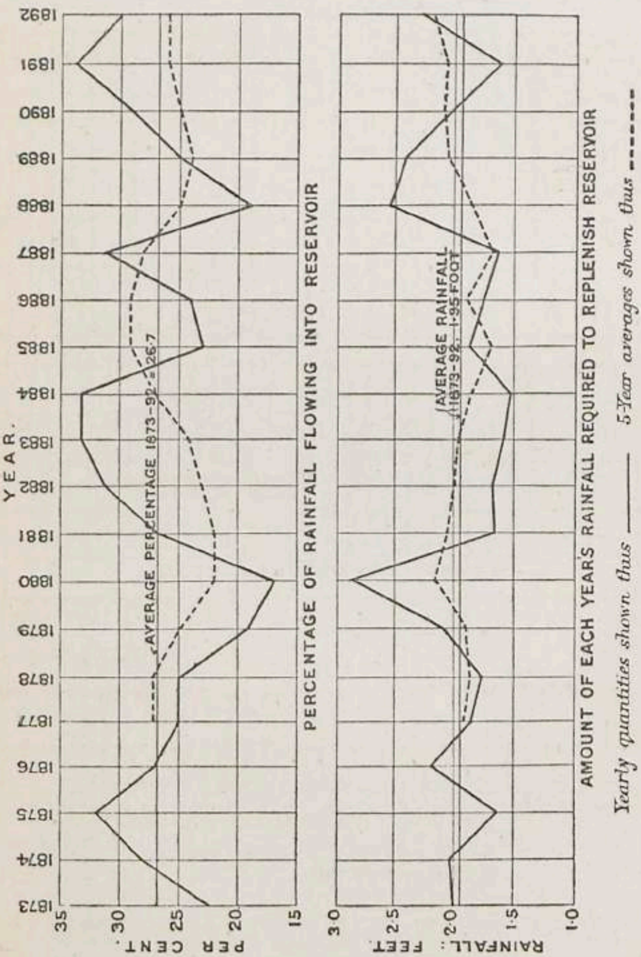
Figs. 42.



COMPARATIVE RAINFALL OF TWO PLACES IN INDIA SERVED BY THE SOUTH-WEST MONSOON CURRENTS.

Figs. 42 and 43, worked out several years ago, when considering additions to the Nagpur waterworks. The diagrams, being self-explanatory, were submitted without further comment than that if some margin of safety was allowed, such data, if available at one place, should enable a successful forecast to be made in connection

with proposed works similarly situated and served. He had not previously considered the possibility of extensive percolation of flowing water through surface soils to underground fissures in granite, as referred to by Mr. Phipps Williams, nor did he think it at all probable that by this means there could be drained away such The Author.



Figs. 43.

Yearly quantities shown thus ——— *5-Year averages shown thus* - - - - -

NAGPUR WATERWORKS. WORKING OF RESERVOIR.

immense quantities of flowing water in all the five basins for which figures had been given in the Paper, while in exactly the same class of country the reservoir itself should prove fully stanch with a head of 100 feet of water over its bed. When questioning the utility of catch-water drains, Mr. Strange had evidently

The Author. considered only the extreme feature of streams draining precipitous country, and liable, therefore, to the heaviest floods. Even in such country a large proportion of the off-flow could be diverted; while the Author's experience in half-a-dozen cases in less precipitous country had amply demonstrated that these drains could be most successfully used in connection with the Coolgardie water-supply reservoir. Moreover it had to be remembered that their aid was required in periods of small yield, and not in bumper years. The maximum flood-discharge anticipated at the Helena weir had been checked by deduction from known floods in adjoining streams, and as there had been settlers in the country for three-quarters of a century, this was a more accurate method than that referred to by Mr. Smith, of forecasting from flows at low states of the river and checking by means of formulas with arbitrary coefficients. The Author's experience was that probable maximum flood-discharges were easier of computation than low, or average, or total discharges. As regarded the minor points raised in connection with the weir, it was stated on p. 16 that the usual temperature and other cracks had duly appeared in the weir. These remarks Mr. Fuertes had evidently overlooked. With reference to the position of spill-water discharge arrangements, Mr. Strange had raised points similar to those of Mr. Deacon in the Discussion, to which the Author had already replied. Undersluices, unless very large, would not have much effect, and large sluices could not be recommended for a position such as that at Mundaring; they would have entailed far more expenditure than they were worth. The position of the lowest draw-off outlet referred to by Mr. Strange was not at the lowest point, which was R.L. 320, but 20 feet above, at R.L. 340, which was a very good allowance for silting in a reservoir 7 miles long, as of course the heaviest silting would naturally occur where the inflowing waters first met those of the lake. The scour was at R.L. 330, or 10 feet below lowest draw-off level. It was of course an exploded idea that a scour-pipe, however large, would denude the bed of a reservoir of silt, or even remove a large portion of it; but on the other hand, the scour-pipe should certainly keep the approaches to the draw-off outlets clear, especially if placed so much below the lowest as in the present case. The question of filtration of the water, raised by Mr. Moncrieff, was very important. At present the catchment was rigidly guarded against pollution; but this might not be possible always, and then filtering would be necessary. He had so warned the

The Author.

Government regarding the similarly-situated supply of the Western Australian metropolitan area. It would be easy to introduce filtering arrangements when necessary in connection with the Coolgardie water-supply, if not immediately below the weir, then along the pipe-line, for instance at Baker's Hill, where there would always be ample head to spare. It might be of interest to note that the water as delivered on the goldfields was 70 per cent. purer bacterially than that at the source. So long, however, as precautions were taken to keep the catchment free from pollution there were no especial dangers; for although, as pointed out by Mr. Hazen, long storage in a warm climate aided in promoting the inferior growths, both animal and vegetable, evidently there were also counteracting influences where catchments were protected, as was shown in the following comparative Table

Month.	Melbourne Tap-Water. Average Number of Bacteria per Cubic Centimetre.	Perth Tap-Water. Average Number of Bacteria per Cubic Centimetre.
April	?	780
May	?	147
June	?	298
July	132	132
August	99	370
September	76	90
October	89	30
November	60	1,572
December	71	3,470
January	54	?

of the average number of bacteria per cubic centimetre. The storage reservoirs of both towns were comparatively shallow; the Melbourne catchment was rigorously protected and the water was unfiltered, but that at Perth was subjected to what was designated a filtering-process. The correspondence elicited by the Paper was especially wealthy in information regarding pipes and pipe-lines. In reply to the queries and inferences contained in the remarks of Messrs. Fuertes, Goument, John W. Hill, List and Strange, the Author would first state generally that the desirability of using wood-stave, ferro-concrete or cast-iron pipe in lieu of steel was duly investigated before preference was given to the last mentioned. As regarded the wood-stave pipes Mr. Fuertes had himself pointed out a very serious objection, namely, that they should always be kept full, otherwise rapid decay must result: a glance at the longitudinal section of the Coolgardie main showed the impossibility of keeping it fully charged

The Author. without continuous pumping at the maximum rate. Having recently returned from the International Railway Congress held at Washington, and subsequent inspection of works under the auspices of the American Railway Association, the Author had nothing but admiration for the directness of American engineering aims; but, on the other hand, it was his opinion that American estimates of cost, especially comparative figures, were not of overmuch value for comparison in other countries, unless full allowance was made for the fact that, so far as he could judge, American home works paid more for materials, etc., than was charged for goods intended for export. Thus Mr. Fuertes stated that the tender for $\frac{1}{4}$-inch steel pipe, 30 inches in diameter, for work at Lynchburg, Virginia, amounted to 16*s.* 5*d.* per lineal foot in position. It was not quite apparent to the Author whether the cost of valves and specials, culverts and aqueducts, meters and road-crossings, covering-pipes and contingencies, were or were not included in this figure. Trench excavation was not included, and if, for the purposes of comparison, the cost of this item and also carriage by rail from the Western Australian sea-coast inland were deducted from the Coolgardie water-supply figures for identical pipes, the net cost arrived at was 16*s.* 6*d.* per foot, inclusive of all the incidental works above mentioned. He considered that this told strongly against the American figures, having in view the fact that the whole of the plates, bars and lead was imported, a very large portion from America itself, and the further fact that wages in that country were 25 to 30 per cent. below those of Western Australia. On the question of the comparative cost of cast iron, raised by Mr. Goument, the Author would direct attention to Mr. John W. Hill's experience at Philadelphia. In the neighbourhood of cheap iron, Mr. Hill found that 60-inch cast-iron pipes, to withstand the low head of 160 feet, cost £4,650 per mile more than steel pipes would have been obtained for. Apart from this, neither cast-iron nor ferro-concrete was in the Author's opinion suitable for the high pressures of the Coolgardie water-supply. These could no doubt have been reduced by introducing more pumping-stations, but unprofitably, owing to the enhanced cost of pumping. The actual life of steel pipes, raised by Messrs. Goument and List, was another question, but it did not enter too largely into the pipes dealt with in the Paper, for, as therein stated, a sinking fund of 3 per cent. per annum on the whole cost of the works has been provided by Act of Parliament to discharge the loan at a comparatively early date. The Author was glad to note

The Author.

Mr. Herbert's good opinion of the locking-bar pipe, but considered that he had rated its virtues somewhat too highly. For the Coolgardie water-supply, at any rate, the price asked, length for length and thickness for thickness, was higher for locking-bar than for riveted pipe. He did not propose to follow Mr. Herbert in his calculations as to the comparative cost and economy of pipes of various diameters, as they appeared to be based on the fallacy that a 31-inch diameter pipe laid complete cost $\frac{31}{32}$ of the cost of a main 32 inches in diameter. This, at any rate, was not the Author's experience. Professor Campbell Brown's statement that asphalt was not suited for use in a composition employed for pipe-coating was too sweeping. The lime contained in the asphalt would no doubt be injuriously affected by some waters, but in numerous cases this material had served excellently, instances being forthcoming in the works mentioned by Messrs. Fuertes and Moncrieff. The coating referred to by Mr. Smith must have been wrongly made or applied, or used in some unsuitable position, to have perished, while exactly similar coating had served well in South Australia, as mentioned by Mr. Moncrieff. The precautions against corrosion suggested by Mr. List would surely prove altogether too expensive for general use, even if they were successful, which was not certain. With reference to the jointing of the pipes, no yarn had been used, the lead being kept from running into the pipe by temporary expansive rings; as the sleeve-rings varied slightly in diameter, the caulking-tools used also varied in size; the caulking-machine was in the Author's opinion eminently suited for use on ordinary socketed pipes; and there was no doubt that a machine suitable for caulking at the locking-bar could be devised and would save money. The cost of caulking in South Australia mentioned by Mr. Moncrieff did not appear to have been lower than on the Coolgardie water-supply when due allowance was made for the larger pipe and for higher wages. On the latter work the cost of hand-caulking was 4*s*. 3*d*. on a 30-inch pipe, equivalent to 3*s*. 8*d*. on the 26-inch pipe used in South Australia. The Coolgardie wages were, per day of 8 hours:—Foreman, 16*s*. 6*d*.; hand-caulkers, blacksmith, lead-melter, etc., 12*s*. 6*d*.; labourers, 10*s*.; and the corresponding rates in South Australia were, according to the Author's notes, 10*s*. 6*d*., 7*s*. 6*d*., and 6*s*. 6*d*.—less than two-thirds on the average. Reducing the Coolgardie figures for 26-inch pipe even to two-thirds, the cost per joint was less than 2*s*. 6*d*., the lower of the South Australian figures; and as mentioned in the Paper, machine-caulking cost 1*s*. less per joint, besides saving in depth of manhole and ensuring uniform

The Author. work. In reply to the concluding portion of Mr. John W. Hill's remarks, the Author had obtained in India all but absolute water-tightness in a new cast-iron main 12 inches in diameter and about 2 miles long. It was more than questionable, however, whether it was worth striving after such a condition of things. But even in the rapid work of the Coolgardie water-supply there were lengths from which the leakage was less than the figures given on p. 27. Thus the $36\frac{1}{2}$ miles of main west of station No 4, gauged statically for 2 days, after being filled for the first time, showed a loss of 119 gallons per mile per diem, and after attention to visible leakage it showed a month later 31 gallons per mile per diem, and another month later 35 gallons per mile per diem, or, say, $1\frac{1}{2}$ pint per joint. The Author concurred with Messrs. Bruce and Hill as to the value of asphalt lining for reservoirs, and he had used it on the Western Australian goldfields for this purpose; but he employed 1 inch of asphaltic mortar without concrete or puddle, and found it sufficient for a head of 15 to 20 feet of water, while the cost of construction was one-fourth to one-third of that of a sufficient thickness of concrete lining. It was not possible to concur in Mr. Marsh's remarks as to the inutility of the wires buried in the concrete lining of the Bulla-Bulling reservoir. This work had been carried out in the height of a hot summer, and the lining must have been subjected to great tensile stress when cooled by the admission of water. One after another of several concrete reservoirs on the goldfields had suffered from cracks all over, and consequent leakage, whereas the Bulla-Bulling lining had opened only at the expansion-joints, as already stated. There was no doubt, as Mr. Fairley said, that the capital cost of the pumping-machinery would have been reduced by employing three sets at each of the last four stations, but this would have meant two sets working at one time instead of one, as in the adopted scheme; and at West Australian rates of wages this would have meant an appreciable increase in the cost of maintenance. Other advantages would also have been lost. Mr. Phipps Williams's calculations on the possible saving in pumping-machinery, if intended to be taken literally, were, the Author regretted to say, beyond him. Was it intended to imply that small pumps cost the same amount per horse-power to instal as large ones, and that working small pumps three shifts costs as little as doing the same amount of work in one shift with large pumps? How also would the increasing demand (increasing until the ultimate estimated amount should be reached) have been provided for with such small pumps as Mr. Williams considered enough? The small pumps, unless of varying

power, could not have been placed at the stations decided on as best for the ultimate supply, and nothing but peripatetic pumping-stations would have permitted of uniformity and economy in size of pumps without excessive pressure on the main as the horse-power rose in keeping with the demand. The details of cost of sundry works referred to by Mr. Moncrieff had not been given by the Author, as he was in hopes that they would be supplied to the Institution later on by his principal assistant, Mr. Reynoldson, to whom the Author, being now engaged in London and unable therefore to comply with Mr. Crowell's suggestion of further tests of friction etc. from time to time, looked for compliance in this direction also. In conclusion he desired to express his obligations to those who had commented so kindly on the work carried out. Very few indeed, besides engineers, credited the intense anxiety entailed during construction of a work of this kind; and none but engineers appreciated the difference between the reasons for success and failure in connection with Colonial works. The Author.